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BEHAVIOR OF THE REBUILT WOLF CREEK CULVERT

Prepared for
Montana State Highway Commission
Planning Survey Section
In cooperation with
U. S. Department of Transportation

ENGINEERING

Research Laboratories



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Behavior of the rebuilt Wolf Creek culve



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In cooperation with
U. S. Department of Transportation
Bureau of Public Roads

by
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1968

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.



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ABSTRACT

The failure of an 18.5 foot diameter structural-plate culvert under 83 feet of cover led to its reconstruction using the "imperfect trench" type of construction as well as other changes. This paper reports the findings of a research project that was initiated to study the behavior of the reconstructed culvert. The study was sponsored by the Montana State Highway Commission and the Bureau of Public Roads and was performed by the Department of Civil Engineering and Engineering Mechanics at Montana State University.

The reconstructed culvert was instrumented to study its behavior. Instrumentation consisted of: SR-4 strain gages placed on the culvert walls at approximately mid-height; Carlson Soil Stress Meters, placed on the outside walls of the culvert and in the fill; rubber pressure cells, which were commercial hot water bottles, placed in the fill; and settlement cells, which were placed in the fill on each side of the culvert. Also, measurements that were taken included: levels on the culvert invert, vertical and horizontal diameters, and levels of the roadway profile.

Computer programs were developed to determine stresses and earth pressures from data obtained by periodic readings of the instruments.

Overall, the instrumentation performed satisfactorily. Results from strain gages, stress meters, and rubber pressure cells, correlated well and demonstrated that the vertical load on the culvert was much less than the weight of the overlying column of earth - a highly favorable load condition which the "imperfect trench" method of construction was expected to produce.

The SR-4 strain gages, which measured bending strains in addition to circumferential compression, showed that significant residual bending stresses were induced in the plates during erection. They also monitored bending stress changes that occurred at the strain gage sites during and after the backfilling operation. The vigorous compaction of the backfill on each side of the culvert, during the early stages of backfilling, produced a bending stress pattern in the side walls that persisted throughout the embankment construction period and thereafter.

There was no measurable differential settlement of the pavement attributable to the "imperfect trench" type of construction.

CHAPTER I

INTRODUCTION

This paper describes the behavior of an 18.5 foot diameter structural-plate culvert that was reconstructed after the failure of the original structure. Construction of the original installation began in the fall of 1963. The rock fill, which provided approximately 83 feet of cover on the culvert, was completed in the spring of 1964. It was discovered that the structure had failed in the summer of 1964, within three months after the fill was completed, but it is not known exactly when the failure began.

Solutions for repairing the structure were reviewed and the final conclusion was to remove the fill and rebuild the failed portion of the culvert. Removal of the fill began that winter. Reconstruction of the culvert was delayed until the following fall, 1965, because of higher than normal runoff and the resulting problem of stream diversion.

Extensive testing by the culvert contractor was done to determine the cause of the failure. The test results indicated that the failure may have been caused by "hydrogen embrittlement" of the zinc-coated high strength bolts. It was thought by some of the engineers involved that the culvert loading may have been larger than anticipated; therefore, the reconstruction was designed for an increased margin of safety. In addition to changing the type and size of bolts, an "imperfect trench"

type of construction was used. It was decided that the culvert should be instrumented and that a study should be made on the behavior of the reconstructed culvert.

The design and installation of instrumentation, and the study of the behavior of the reconstructed structure was performed by the Department of Civil Engineering and Engineering Mechanics at Montana State University. The study was under the sponsorship of the Montana State Highway Commission and the Bureau of Public Roads.

CHAPTER II

BACKGROUND INFORMATION ON THE WOLF CREEK CULVERT

The culvert installation is located on Little Prickley Pear Creek near the south end of Wolf Creek Canyon on Interstate Route 15 about 25 miles north of Helena, Montana. The Interstate plan-profile at the culvert site is shown in Figure 1. The design details of the original culvert are covered in a paper by Kraft and Eagle (8)¹.

The culvert was 548 feet long. Corrugated and galvanized structural steel plates, 3/8 inch thick, were used in the middle 328 feet and thinner plates were used near the ends.

Design loading on the structure was 8.2 kips per square foot from 83 feet of fill having an assumed unit weight of 105 pounds per cubic foot. The use of 3/8 inch plate and 3/4 inch high strength bolts resulted in a design seam strength of 270,000 pounds per linear foot and provided an apparent safety factor of 3.5. The high strength bolts used with the 3/8 inch plates of the original culvert were zinc-coated ASTM A490² bolts while the remaining bolts used with the thinner plates, were zinc-coated ASTM A325 bolts. Because the original structure was not instrumented, the actual load on the structure could not be determined.

The failure was due to "brittle-like" fracture of the bolts along the longitudinal seams of the lower half of the culvert. The failure

¹Numbers in parentheses refer to references listed under Literature Cited.

²The original designation of these bolts was ASTM Designation A354, Grade BD.

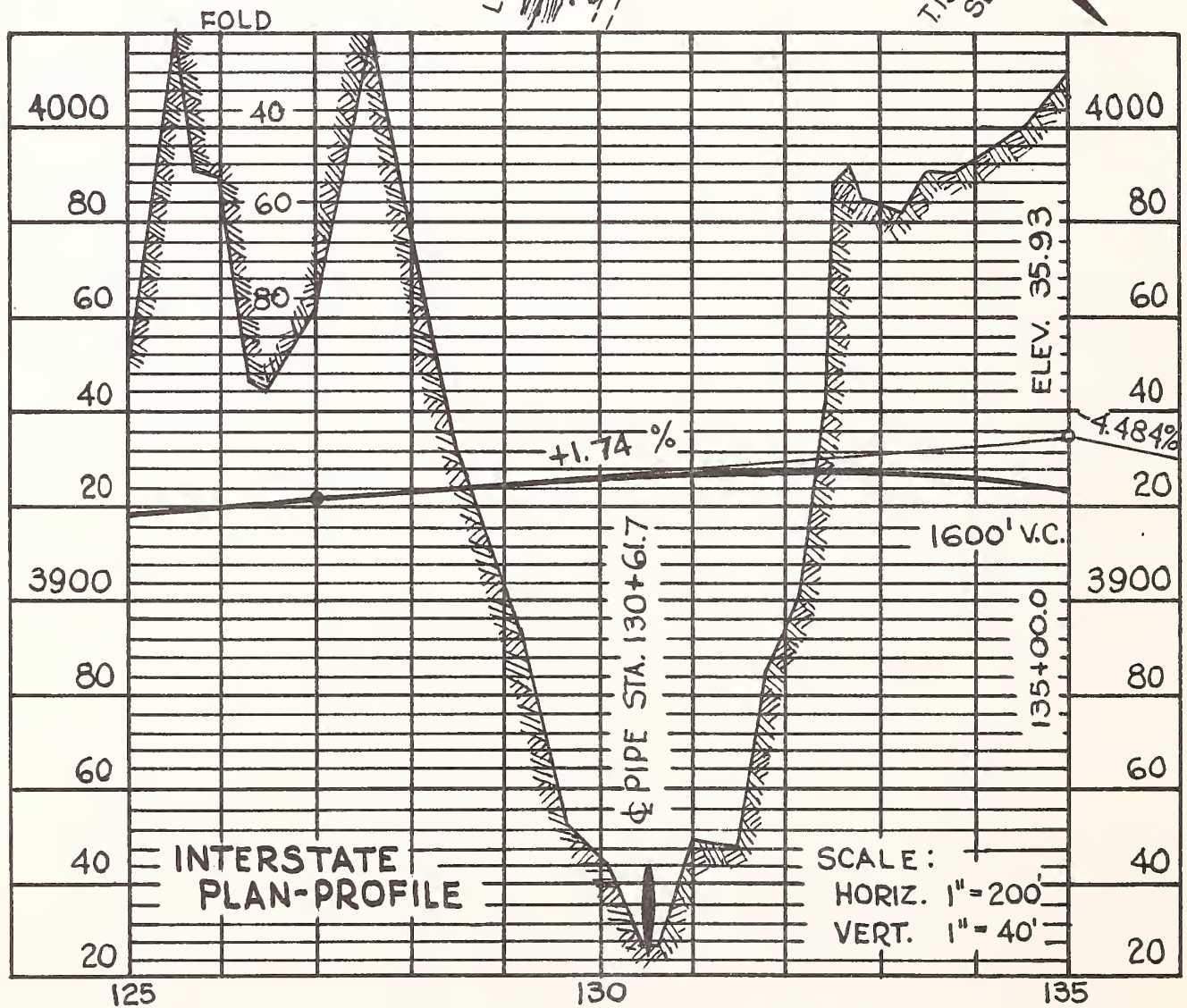
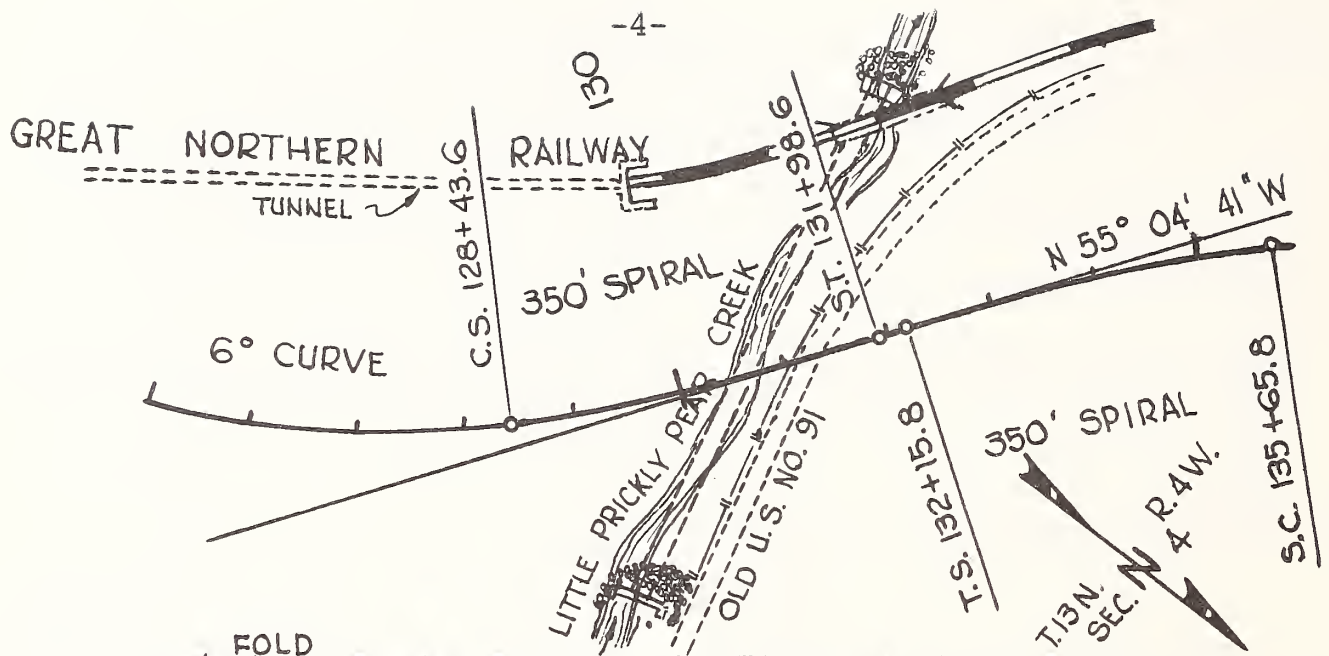


Figure 1. Plan and Profile at Culvert Site

extended throughout that section of the structure using 3/8 inch plate and high strength bolts. Here the plates had slipped over each other, in some cases overlapping approximately seven feet, decreasing the overall diameter to approximately 16 feet. Large ruptures near each end of the failed section allowed fill material to flow into the structure. This influx of material formed a dam near the inlet end, ponding water approximately seven feet deep during a time when the water downstream in the culvert was two to three feet deep.

The research center of Armco Steel Corporation, the culvert contractor, performed extensive testing to determine the cause of failure and the results were presented by Macadam (12). Extensive laboratory static testing of bolted seams, including pure compression and pure bending, showed that the plate, not the A490 bolts, failed at the seam. Because of the "brittle-like" appearance of the failed bolts from the culvert, sections of bolted plate were also tested at extreme temperatures. In one case the temperature was lowered to -320 degrees Fahrenheit by liquid air and static and impact tests were performed, but there were no A490 bolt failures. This testing program showed that the bolts were mechanically sound.

Extensive tests involving an electrochemical mechanism were conducted because high-strength steels are susceptible to hydrogen embrittlement and hydrogen-stress cracking. This program showed that A490 bolts could fail by hydrogen embrittlement within 15 minutes under normal tension

loading and certain extreme environmental conditions. A325 bolts tested under the same conditions were not affected by hydrogen embrittlement. Macadam's report supported the hypothesis that hydrogen embrittlement was the cause of the bolt failures in the culvert and prompted a decision that larger bolts of a milder steel would be substituted for all high strength bolts in the reconstructed culvert; specifically, 7/8 inch A325 bolts were used in place of the 3/4 inch A490 bolts. Specifications for the rebuilt culvert also stipulated that the bolt tension induced by nut tightening should be not less than 18,000 nor more than 22,000 psi, (pounds per square inch). This corresponded to a torque of approximately 200 foot-pounds.

CHAPTER III

THEORY OF CULVERT LOADING AND THE IMPERFECT TRENCH

"Marston's Theory of Loads on Underground Conduits" (13) was one of the first practical methods, using principles of mechanics, to estimate the load on an underground structure. There are several different ways in which conduits are placed in the ground, such as in tunnels, in ditches, or by placing fill over the conduit. The latter two methods have been divided into three main classes by Marston. These classes are the "ditch conduit", the "negative projecting conduit" and the "positive projecting conduit". The original and reconstructed Wolf Creek culverts, like most highway culverts, were installed as positive projecting conduits.

A positive projecting conduit is one that is placed on the ground surface (not in a narrow trench) and then covered by fill material. The vertical soil load acting on such a conduit may be smaller than, equal to, or larger than the weight of the vertical-sided column of soil located directly above the conduit, depending primarily upon whether that column settles more than, the same as, or less than the adjacent columns of earth.

An extensive program of research has developed from Marston's work on underground conduits (13, 14) which he began in 1908 at the Iowa Engineering Experiment Station. Independent experimental work accomplished in the 1920's at the University of North Carolina, by Cain et al (4); and by the American Railway Engineering Association, Farina, Illinois, (2)

agrees with Marston's work. Recently, with the advent of high fills in Interstate Highway design and buried structures for blast protection, there has been more experimental research on the loading of underground conduits. Some of these works (5, 6, 23) accept Marston's theory of loading. Others (1, 3, 11) approach the subject from a different soil-structure interaction concept; however, this research has not been completed to the extent that it is available for practical application.

The imperfect trench method of construction proposed by Marston to relieve earth pressures on positive projecting conduits under certain soil conditions was also advocated and tested by Spangler (19, 20) who worked under Marston and continued Marston's work. The imperfect trench method is based on the theory that if highly compressible material is placed above a conduit the column of soil above settles more than the adjacent soil on each side. This induces "arching" so that the column of soil is partially supported by the adjacent soil and therefore transmits, to the conduit below, a load which is smaller than the weight of the column.

Marston recommended a compressible material such as loose fill material, or even leaves or straw, be used as fill directly above the structure. This material can be placed by compacting the fill to any desired height, trenching along the center-line of the culvert and then backfilling with the loose material; or by placing bales of straw along the centerline

of the culvert at any desired height and compacting the fill adjacent to and above the straw. This latter method was used for the rebuilt Wolf Creek culvert. Figure 2, a typical cross-section of the rebuilt culvert, shows the location of the baled straw.

The imperfect trench type of construction was not readily accepted by some engineers because of the fear of unequal settlement of the surface of the fill. Detrimental unequal settlement at the surface is not expected to occur at the Wolf Creek culvert because of the high fill and because of an eight-month delay in paving the road. In high fills a plane of equal settlement will usually develop within the fill and there will be no differential settlement at the surface. Unfortunately, in most practical situations, the minimum fill height needed to prevent differential surface settlement cannot be accurately predicted.

Recently, on large culvert installations involving both rigid and flexible pipe, the imperfect trench method of construction has been used with varying degrees of success. Timmers (22) reports the use of an imperfect trench for three 84 inch diameter corrugated metal culverts installed under a 137 foot fill in Cullman County, Alabama. Costes and Proudley (6) also report the use of the imperfect trench for a 66 inch diameter corrugated metal culvert under a 170 foot fill in McDowell County, North Carolina. The method of backfilling a trench with loose earth was used for these two installations. This method did not prove to

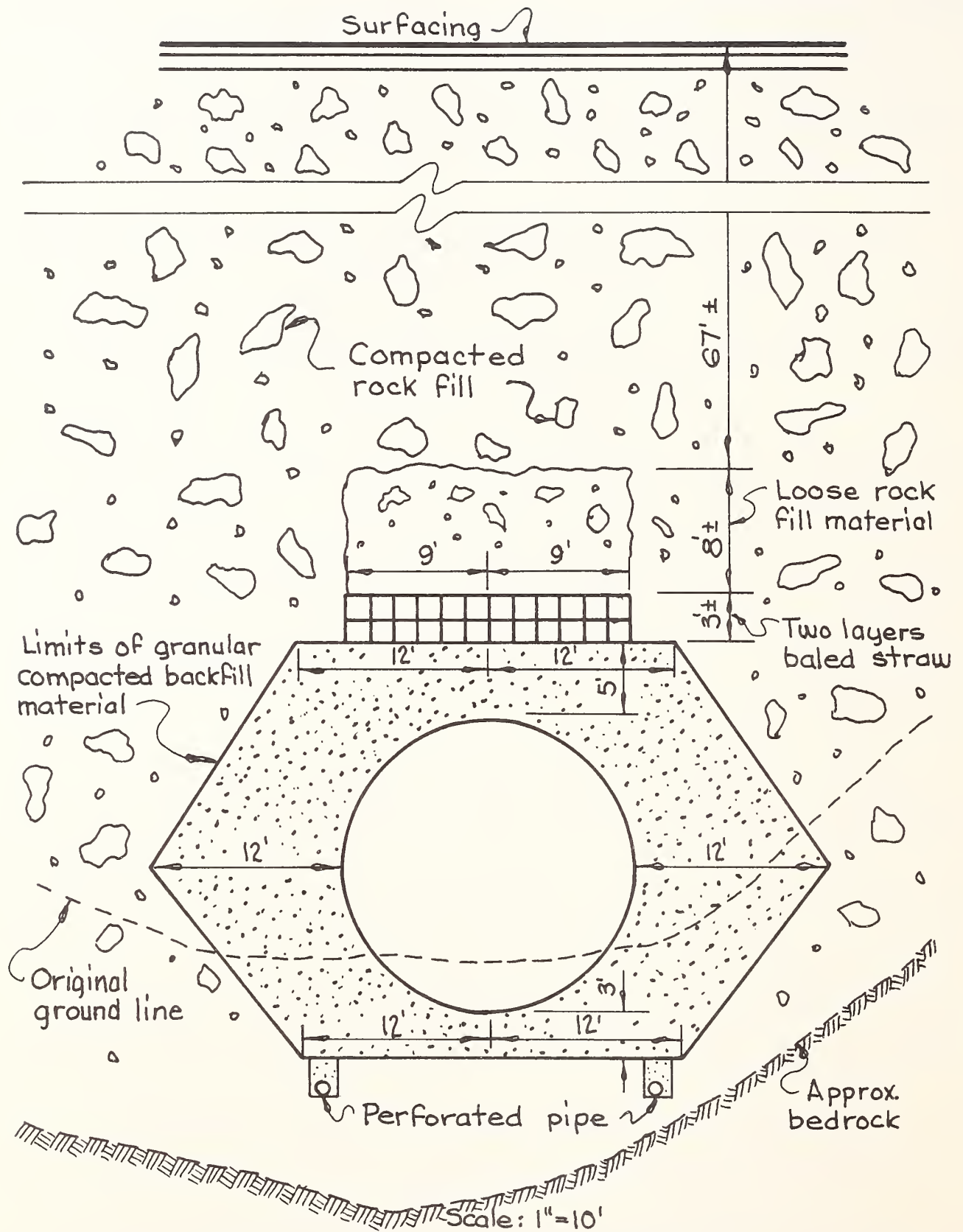


Figure 2. Typical Cross-Section of Rebuilt Culvert

be as successful as anticipated, but the loading on the pipes was generally not greater than the weight of the column of earth above the pipes and many load measurements were lower. Larson (9) reported on three reinforced concrete culverts, under fills 38 to 65 feet in height, that were installed in Humboldt County, California, using baled straw as the compressible medium. These structures performed better than those installed without straw. Conclusions of the work in California were that fill heights about three times that allowed by normal methods could have been used without overloading the pipe.

CHAPTER IV

SELECTION AND DESIGN OF INSTRUMENTATION

One of the primary objectives of the research project was to determine the earth pressure acting on the rebuilt Wolf Creek culvert. This pressure was determined by the direct method of measuring the earth pressure acting on the culvert and also by the indirect method of determining strains in the culvert and calculating the earth pressure required to produce those strains .

Commercial instrumentation, available on short notice, was used because of the possibility that the contractor would begin reconstruction of the culvert at an early date. One of the problems was arranging the work in such a manner that when construction began the instrumentation could be placed without delaying the contractor .

Carlson soil stress meters, which have proved to be reliable and accurate in other installations, were selected to measure earth pressure. A few hot water bottles connected to direct reading Bourdon pressure gages were also used. SR-4 strain gages were used to determine axial circumferential strains and bending strains in the side walls of the culvert. Settlement cells, which work on a water level principle, were used to measure settlement of the fill adjacent to the culvert. Measurements of horizontal and vertical diameters were made directly with graduated rods, and culvert floor settlement was obtained from periodic direct observations

with an engineer's level.

Strain Gages

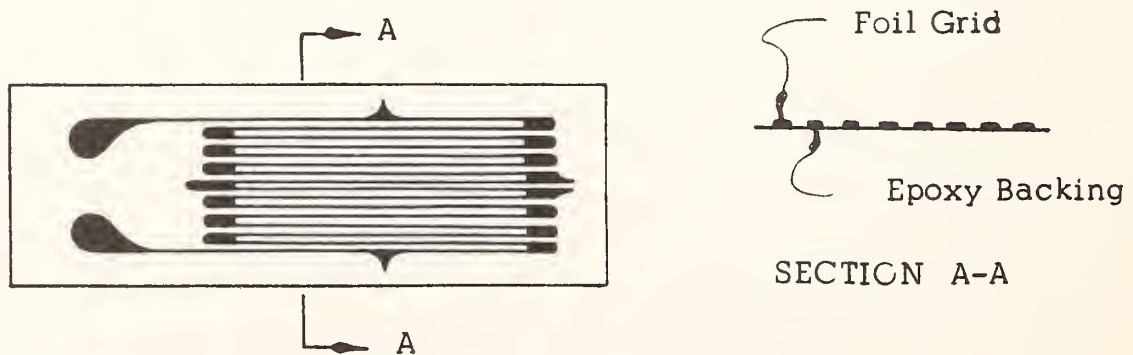
SR-4 Baldwin strain gages were used to measure circumferential strains in the culvert. These gages operate on the principle that the electrical resistance of a wire varies with the tension applied to the wire. By measuring this change in resistance the strain can be determined. This change in resistance, which can be quite small, in the range of a thousandth of an ohm, can be measured accurately by a Wheatstone bridge circuit. If the wire is properly cemented to a material, then strain in the material can be measured by measuring the change in resistance of the wire.

There are many items that should be considered when using strain gages, such as the type strain being measured, the type material that is being strained and the environmental conditions the gage will be subjected to. One excellent reference for the proper uses of various types of strain gages is "The Strain Gage Primer" by Perry and Lissner (15).

The SR-4 strain gages selected for this project were epoxy backed, type FA-100-12. This type of gage is a metallic-foil strain gage which offers a number of advantages over the older paper backed wire strain gage. One primary advantage, because of its epoxy backing, is its superior long-term stability under adverse environmental conditions. This

gage is also made from thin strain-sensitive foil which offers other advantages. The foil gage can be made so that the width of the foil at the end of each loop can be increased to reduce the sensitivity to transverse strains, and the cross-section of a gage can be small in relation to its surface area so that the cement holding the gage is considerably stronger than the foil. The increase in width at the end of each loop and the cross-section of a gage is shown in Figure 3. The gage factor, which indicates the sensitivity of a gage, is higher for the foil gage than the equivalent wire gage.

Careful attention must be given to properly attaching the strain gages to assure proper functioning. It was known that the gages would be in use over an extended period of time and more than likely would be subjected to severe moisture conditions. Also, the gages had to be placed on the plates in the field where conditions were more severe than in the laboratory. The cement selected was Eastman 910 cement because it would cure at relatively low temperatures and had a very rapid curing time.



Not to Scale

Figure 3. Foil Strain Gage

Strain gages must be located properly if useful readings are to be obtained. SR-4 wire strain gages were used on a 66 inch diameter corrugated metal culvert in North Carolina and the results were reported by Costes and Proudley (6). Strain gages were located circumferentially on the geometric neutral axis of the corrugations at the mid-height and the top and bottom of the culvert. Results from this installation were disappointing and some possible reasons for this were:

1. The strain gages that were placed on the bottom could have been grounded, although they were waterproofed and protected.
2. Local buckling may have caused excessive strain at the location of a strain gage.
3. Strains other than uniaxial circumferential strain may have been introduced because of earth movements and unorthodox construction practices.
4. The strain gages were installed at the geometric neutral axis of the corrugations. A slight error in gage location would cause the measurements to include some bending strain.
5. The metal may have been stressed beyond its elastic limit where the linear stress-strain relationship would no longer be valid. This non-linear stress would also cause a shift of the neutral axis.
6. It would be difficult to evaluate the state of stress from a single unidirectional strain measurement even with all stresses below the

elastic limit.

SR-4 foil strain gages were used quite successfully during testing of corrugated aluminum alloy culverts as reported by Koepf (7). Many of the problems encountered in the North Carolina research were eliminated by better placement of the gages on the aluminum culverts. Two groups of six gages were placed at the quadrant points (top, bottom, and each side) for each test section. Each group of six gages was placed with a pair on the inside crest, on the inside valley and at the geometric neutral axis of the corrugations. The gages were protected from moisture but running water was not a problem because the culverts were test structures and not drainage structures. All of the gages were placed on the inside of the culvert. From the stress-strain relationship, the stress at the extreme fibers could be determined by algebraic proportion, and the average compressive stress and maximum bending stress determined.

The strain gages used on the Wolf Creek culvert were placed similarly to those reported by Koepf (7) except that gages were also placed on the outside of the culvert. Six strain gages were installed to measure circumferential strains on each instrumented plate as shown in Figure 4. The gages were placed in pairs, on opposite sides of the plate, at three locations; at the neutral axis of the corrugations and at the bends on each side of the neutral axis. By placing the gages in these positions, the

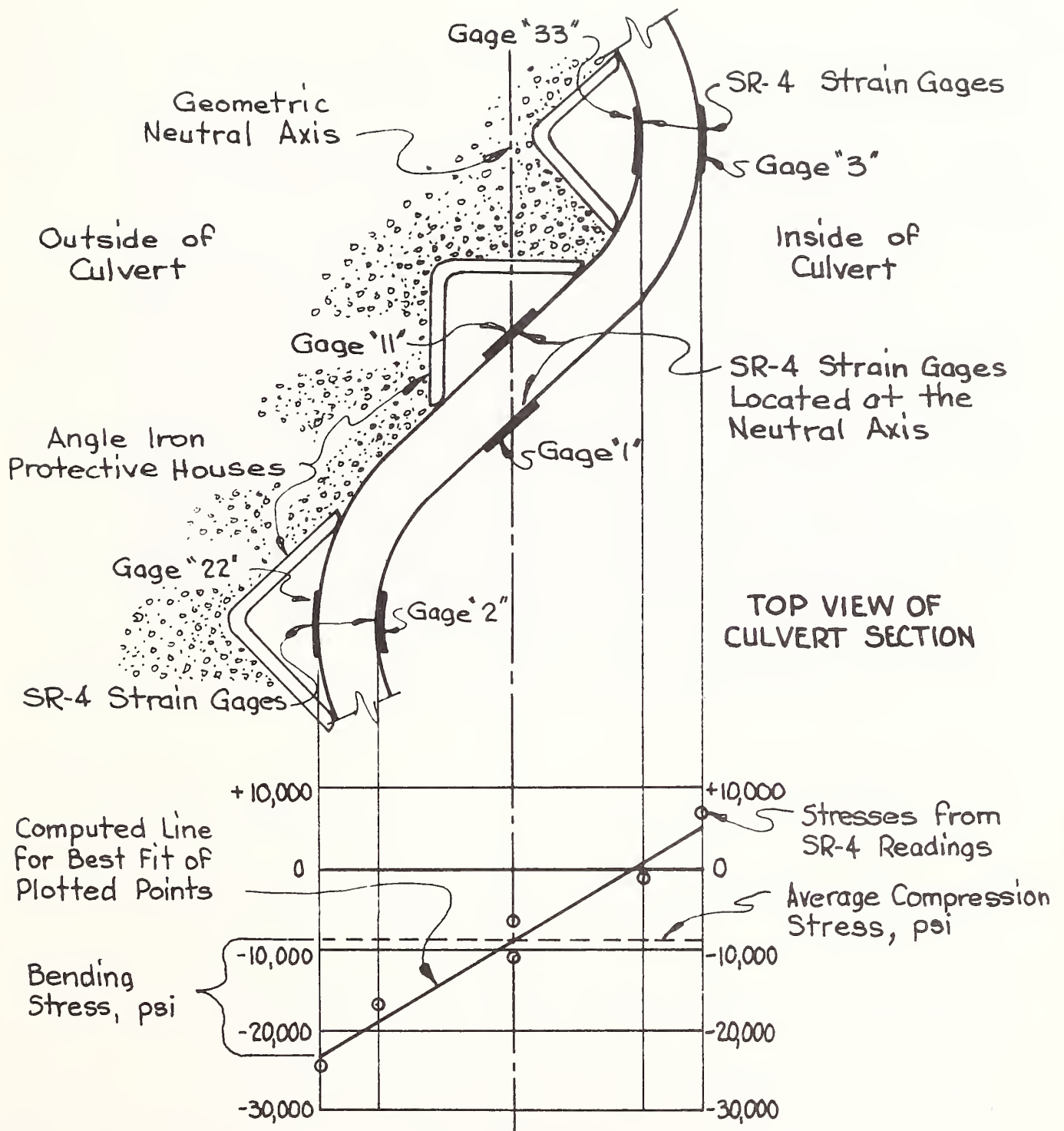


Figure 4. Position of SR-4 Strain Gages to Measure Circumferential Strain in the Corrugated Plates

bending moment could be computed and the average compression determined as indicated in Figure 4. Six separate plates were instrumented utilizing a total of 36 measuring gages.

Additional gages were cemented to small galvanized plates and used as compensating gages. The compensating gage acted as a known resistance when measuring the unknown or changing resistance of the gage attached to the culvert plate. The change in resistance, between any measuring gage and the compensating gage, is proportional to the load-induced strain in the culvert at the location of the measuring gage. The compensating gage also provides temperature compensation because it is placed inside the culvert near the measuring gages and under essentially the same environmental conditions.

Lead wires of 18 gage stranded copper wire with teflon insulation were attached to the strain gages. Lead wires 15 feet long were used in order to eliminate possible strain errors associated with the use of longer lead wires. This arrangement necessitated the use of a portable strain indicator that could be carried inside the culvert to the location of each instrumented plate. A Baldwin-Lima-Hamilton Type N portable strain indicator, which indicates the strain change directly in units of micro-inches per inch, was used in the study. Figure 5 shows the portable strain indicator and also two compensating gages, cemented to small galvanized

flat plates with lead wires attached, before water-proofing.

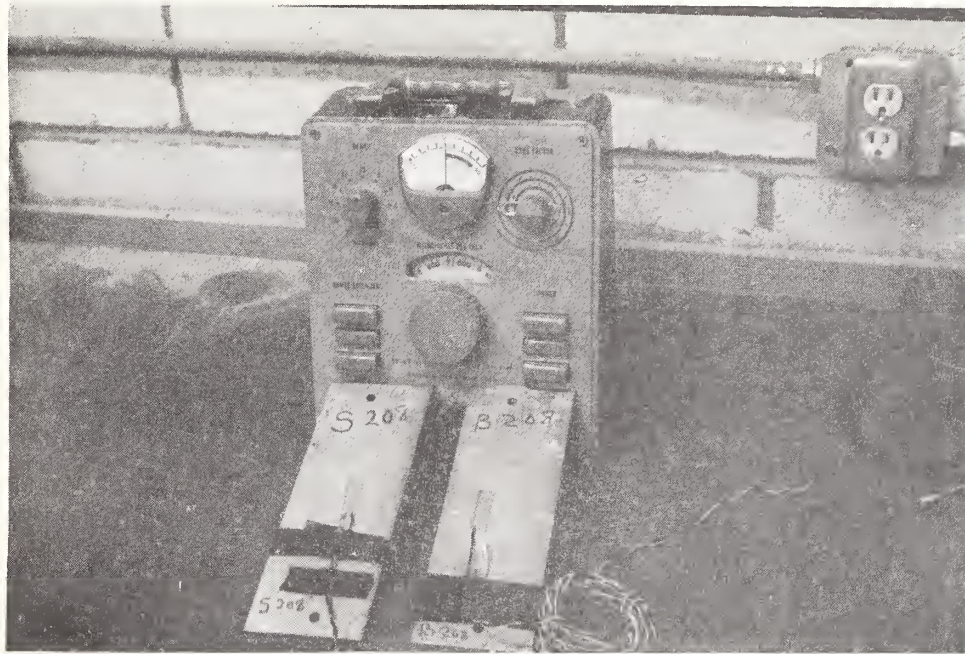


Figure 5. Baldwin-Lima-Hamilton Type N Portable Strain Indicator with Compensating Gages Before Waterproofing

Pressure Cells

Earth pressures can be measured by many methods; some of which are covered in "Foundation Engineering" by Leonards (10). The commercial pressure cell selected to measure earth pressure on the Wolf Creek culvert was the Carlson Soil Stress Meter illustrated in Figure 6. Previous studies have shown this stress meter to be dependable and accurate. It was also selected because it was readily available allowing installation without disrupting the contractors construction schedule. The meter selected measures pressures up to 150 pounds per square inch, is 7 1/4 inches in diameter, one inch thick, and has a stem on the back which

houses the wire resistance coils.

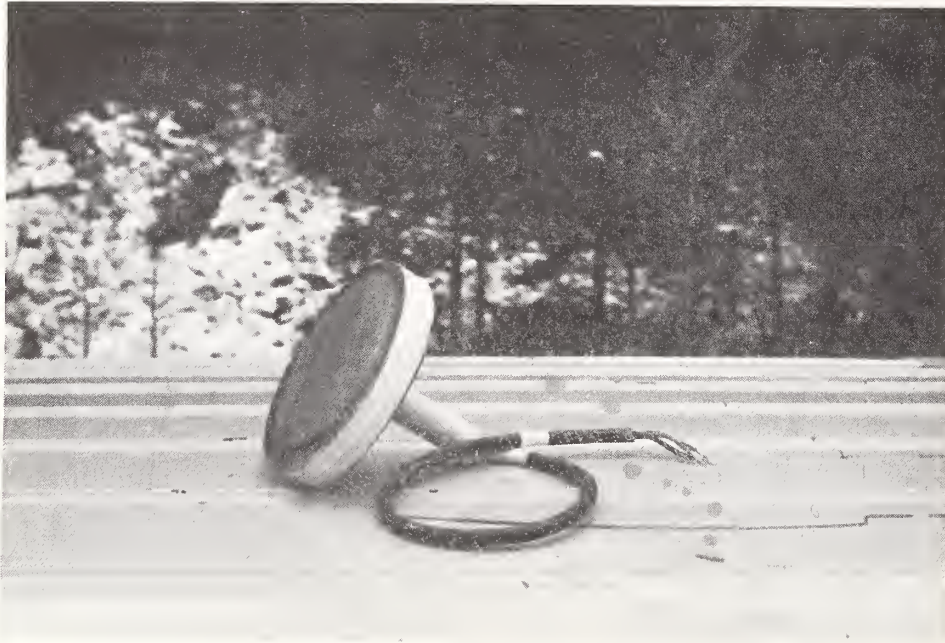


Figure 6. Carlson Soil Stress Meter

In 1943 Terzaghi (21) used pressure cells to measure the contact pressure between the bottom of the invert and the underlying soils during and after the construction of the Chicago Subway Tunnels. With regard to the Carlson stress meter, Terzaghi reported:

Because of the wide previous use of Goldbeck cells, it was considered desirable to make an installation of these devices in the invert section and to compare the readings by means of the Carlson cells and the strain gage readings in the ribs All the Goldbeck cells

remained in operation and gave fairly consistent results. However, the individual readings were so scattered that they gave no indication of the true distribution of the contact pressure as disclosed by the Carlson cells

A comparison of the results of the use of the pressure cells and strain gages used by Terzaghi is shown in Table 1.

TABLE 1. STRESSES IN THE CHICAGO SUBWAY TUNNEL WALLS DETERMINED FROM PRESSURE CELLS AND STRAIN GAGES

Method	Stress in Kips Per Foot
Strain gages, assuming that liner plates act with the ribs	66.0
Carlson Cells	66.0
Goldbeck cells	106.5
Computed overburden, 118 lb/cu. ft.	72.6

The Carlson meter was used quite successfully on the Garrison Dam Project as reported by Sikso and Johnson (18), where 26 of 30 cells installed were operating after 11 years of service. These cells were installed under the powerhouse and the stilling basin with the cell in concrete and the face flush with the bottom of the footing and bearing against the foundation soil.

In a recent review of stress and stain measurements in soil, Selig (17) found that the Carlson meter was one of the most successful used in any of the reviewed field tests. One disadvantage of the Carlson cell is

the large protrusion on the back in which the resistance wires are housed. A modified cell, which is more compact, has been developed and reportedly yields better results.

The stress meter has a pressure sensitive face which transmits pressure by the use of mercury to a flexible diaphragm. The diaphragm is connected to a movable bar on which two ceramic spools are mounted as illustrated in Figure 7. Adjacent to the movable bar is a fixed bar on which two ceramic spools are also mounted. Wire is wound under tension between a fixed spool and a movable spool producing two separate wire resistance coils. The ceramic spools are mounted in a position such that when pressure causes a movement of the diaphragm the tension increases on one coil and decreases on the other.

The use of two coils produces a more sensitive instrument and decreases the effect of temperature. Temperature does affect the readings but temperature corrections can be applied by measuring the resistance of the coils. The change in resistance caused by pressure and temperature is measured by a Wheatstone bridge.

The shape of the meter, particularly the protrusion on the back, created difficulties in mounting it on the corrugated metal culvert wall. The Carlson meter is normally mounted in a rigid wall or footing with its sensitive face flush with the wall surface and against the soil. In

CARLSON STRESS METER

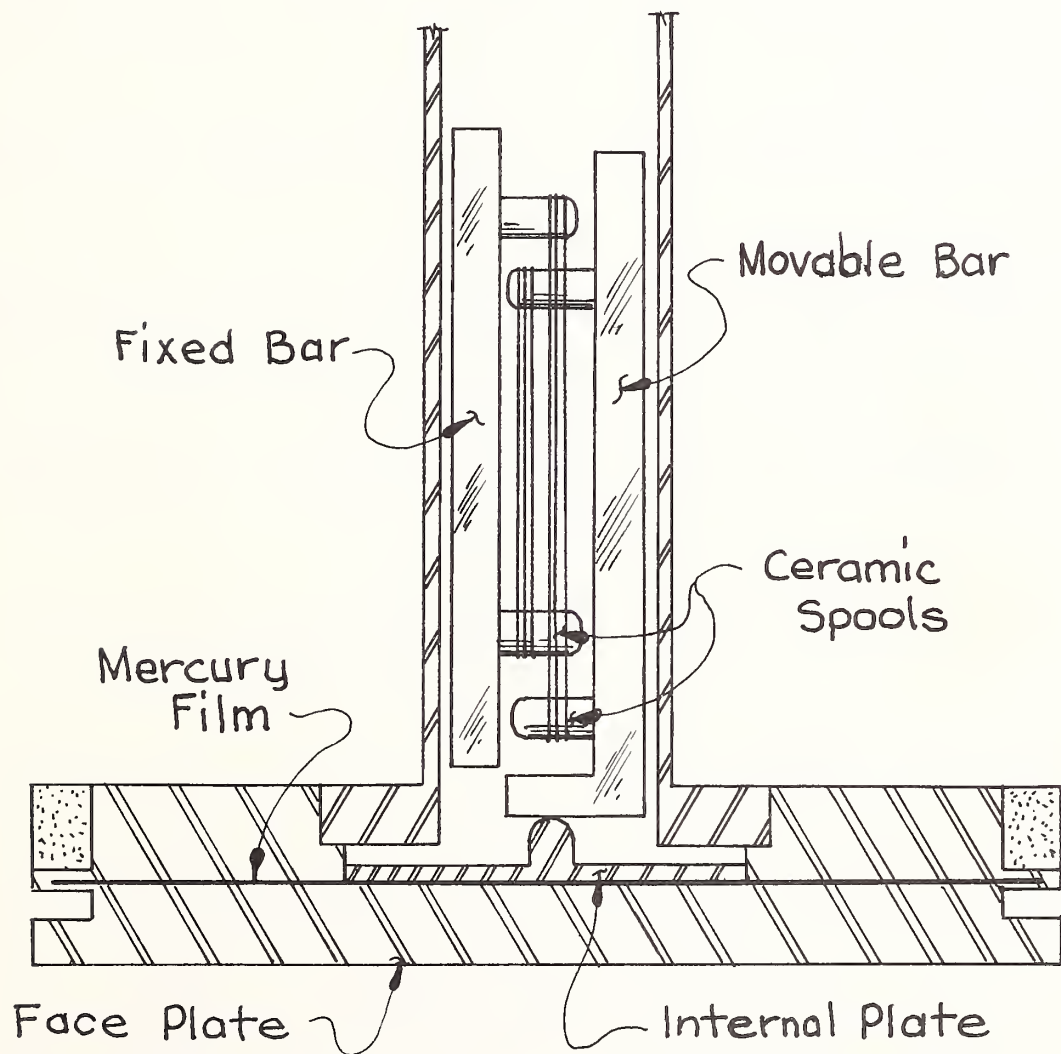


Figure 7. Carlson Stress Meter Cross-Section

this case a mounting frame was made to provide a rigid backing for the cell and to prevent bending moment across the culvert corrugations. Figure 24 on page 45 shows a mounting frame in place on the culvert wall. The backing for the mounting frame was made from a $3/16$ inch thick steel plate, 18 inches square, with a $1\ 3/4$ inch diameter hole in the center for the stem. A one inch thick piece of exterior plywood with a $7\ 1/2$ inch diameter hole was attached to the steel plate to keep the sensitive face flush with the mounting frame surface. A section of $1\ 3/4$ inch pipe, long enough to protect the stem of the cell, was attached to the back of the mounting frame with epoxy cement. Two holes were drilled 12 inches apart, to fit the culvert corrugations, through which the frame was bolted to the culvert.

The stress meters and their calibration data were checked in the laboratory by repeatedly loading the meters to a stress of 100 pounds per square inch in a small testing machine. The indicator used to balance the Wheatstone bridge circuit was a standard stress meter testing set supplied by the manufacturer of the stress meters.

It was known that there would be shearing stresses of substantial magnitude acting on the faces of many, if not all, of the stress meters on the sides of the culvert. This was anticipated from the start, and experiments were conducted in the Civil Engineering Laboratories at

Montana State University, several months prior to culvert erection, to ascertain the effects of shearing stresses on the operation of the meters. Meters were loaded, face down, in a compression testing machine as shown in Figure 8, and then jacked horizontally until slippage occurred between the face of the meter and the bed of the testing machine. The bearing between the wood block and the side of the meter simulated the bearing between the side of the meter and the plywood mounting frame when shear stresses on the face of a meter push it hard against the side of the mounting hole in the plywood part of the mounting frame. From the laboratory tests it was concluded that shearing stresses on the face of a meter were not likely to cause an error of more than 5 percent in the meter output, which was considered acceptable.

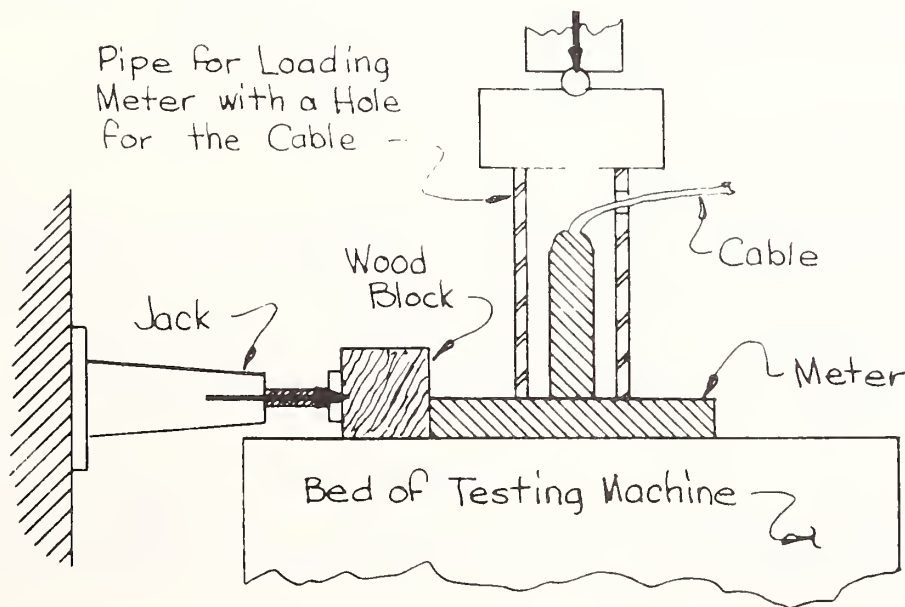


Figure 8. Checking the Effect of Shear Stresses on Stress Meters

Earth pressure can also be measured by the use of a rubber bladder, such as a common hot water bottle, filled with a relatively incompressible fluid, which is connected to a Bourdon-tube pressure gage. This cell was used to determine earth pressures acting on aluminum culverts and was reported by Koepf (7) who was pleased with their performance and stated that predicted pressures compared well with measured pressures.

Six rubber bladder pressure cells of the hot water bottle type, utilizing a mixture of water and antifreeze, were used to measure pressure under the baled straw at the Wolf Creek culvert site. Holes were drilled in the bottle stoppers and copper fittings attached with epoxy cement. Flexible copper tubing ran from each bottle to a station inside the culvert where the tubing terminated in a bleeder valve, a shut-off valve, and a 100 psi (pounds per square inch) capacity Bourdon gage.

Settlement Cells

Settlement of the soil adjacent to the Wolf Creek culvert was determined by the use of liquid level settlement cells. Figure 9 shows a settlement cell and manometer details. The settlement cells were connected to a manometer board with tubing which contained a fluid. Any change in elevation of the cell was indicated by the fluid level in the manometer standpipe. The use of this type of settlement cell was reported by Schlick (16) during work on negative projection conduits.

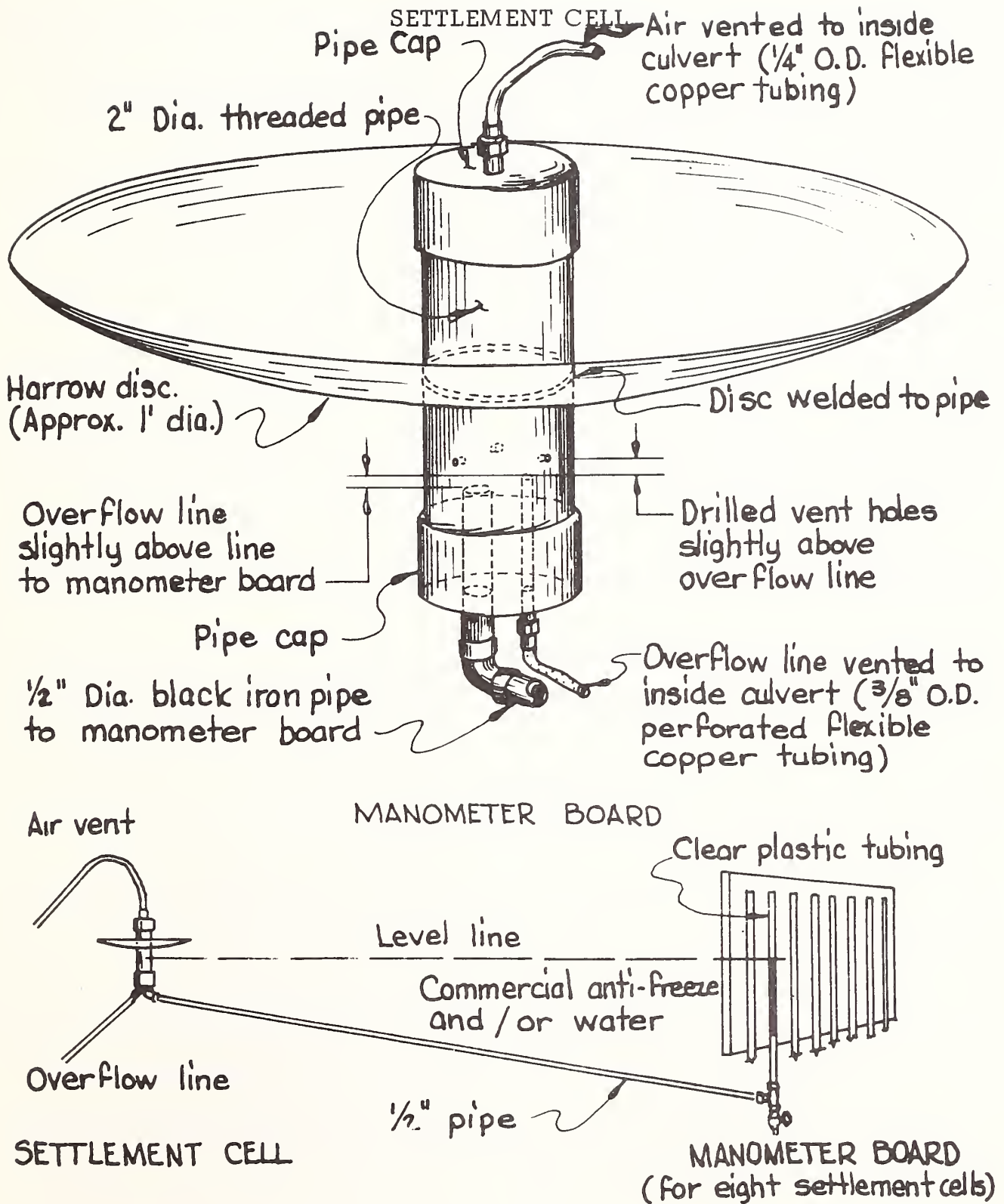


Figure 9. Settlement Cell and Manometer Board Details

The use of a similar cell was reported by Costes et al (6) but a complicated mercury manometer device was used because the readings were taken in the culvert at a point lower than the settlement cells in the fill.

The settlement cells were fabricated at the University instrument shop, from harrow discs and two-inch, threaded pipe. A short section of pipe was welded inside the harrow disc. This section of pipe was threaded on each end and capped. At the lower end two connections, one for a 3/8 inch, flexible, copper line and one for a 1/2 inch outside diameter, black, iron pipe, were connected to standpipes inside the 2 inch pipe. The flexible, copper line was connected to the standpipe that was slightly higher than the other and was used as an overflow line. The 1/2 inch pipe was connected to the lower standpipe and was used as the line to the manometer board. The connection at the top was vented to the inside of the culvert to insure atmospheric pressure in the settlement cell. The overflow line was also vented to the inside of the culvert and was perforated to insure drainage of the overflow. Drill holes were placed in the 2 inch pipe section slightly above the level of the overflow standpipe in case the overflow line could not drain. The manometer board consisted of a large sheet of painted exterior plywood to which 8 sections of clear plastic tubing were connected to indicate the liquid level in each of the 8 settlement cells.

CHAPTER V

RECONSTRUCTION OF CULVERT AND INSTALLATION OF INSTRUMENTATION

The instrumentation was installed while the culvert and fill were under construction so the two topics are very closely related. However, the main points of the culvert reconstruction operation will be reported first in order to provide a general perspective of the entire project before concentrating on the installation details of each set of instrumentation.

Culvert Reconstruction

Following the removal of the rock fill from the damaged culvert in the winter and spring of 1965, disassembly of the failed portion was begun and steps were taken to divert the stream. Unprecedented high water thwarted all stream diversion attempts until September when a successful diversion was finally effected through the use of two diversion pipes, 54 inches and 72 inches in diameter. Figure 10 shows the diversion dam and the inlets of the two diversion pipes, with the undamaged inlet end of the large culvert visible in the background. Both of the undamaged end sections, having a combined length of over 200 feet, were left intact. Only the failed central section was rebuilt.

The intact end sections are visible in Figure 11 which also shows the trenches in which perforated drain pipes were placed to dewater the foundation in the reconstruction zone. The perforated pipes were continually pumped, during culvert erection, because there was no way to



Figure 10. Stream Diversion Dam and Diversion Pipes



Figure 11. Trenching for Perforated Pipe Drain
(Looking Downstream)

drain them by gravity. The culvert bedding was high quality granular backfill material and it was prepared according to specifications which called for the bed to be cut to fit the culvert curvature. The contractor made a steel template to match the culvert curvature and attached it to a patrol blade to cut the bedding. Four inches of loose material was then hand placed to fit this curved bedding. The bedding was completed in the middle of October, 1965.

Disassembled plates were stockpiled in an accessible area downstream from the culvert. Special provisions allowed the re-use of undamaged plates in which the bolt holes were accurately and concentrically reamed, without damage to the plates, to a hole size not larger than one inch in diameter. It was also required that all reamed holes be painted with a zinc oxide primer for a thickness of two to three mils. All plates determined as unsatisfactory by the State Inspector were replaced by new plates rolled to the same radius as the original plates.

The bottom plates were installed first, between the intact end sections. Then the culvert contractor erected plates from each end toward the middle as shown in Figure 12. Erection of the culvert was essentially complete by the beginning of November and control devices were installed to check movement or rotation of the culvert during backfilling operations. Each control device consisted of a 3/4 inch rod,

3 1/2 feet in length, welded to a base plate which was bolted to the culvert floor. A plumb bob was suspended from the roof directly above the rod. The rod was protected from gravel, boulders, and other debris, by a section of corrugated, metal pipe. This section of pipe was 15 inches in diameter and three feet in length and was welded to the base plate. The control devices also served as sites for vertical diameter measurements and floor settlement measurements. One of the control devices is visible in Figure 13 which also shows part of a walkway that was constructed inside the culvert.

While the culvert was being erected, and with the contractors cooperation, a walkway was constructed on the south side of the culvert from the inlet to the downstream end of the 3/8 inch plate. A walkway was also constructed on the north side, along that section of culvert composed of 3/8 inch plate, and crosswalks were placed at each station where vertical and horizontal dimensions and elevations were to be taken. Tables that would fold up against the culvert were placed at each area where instrument readings were to be taken. The walkways, crosswalks and folding tables can be seen in Figure 14.

Backfill work started the second week of November, 1965. The backfill adjacent to the culvert was a crushed granular material of base-course quality and was placed and compacted in layers approximately 6



Figure 12. Pipe Erection (Looking Downstream)

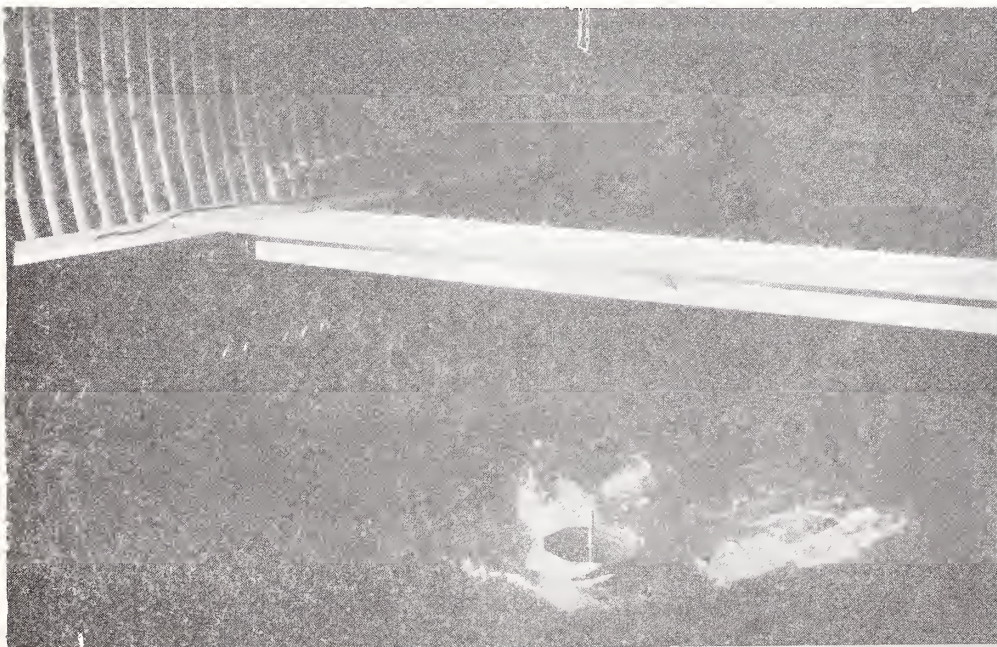


Figure 13. Control Device with Stream Flow Through Culvert

inches thick. The compaction and gradation specifications were as follows:

All backfill material shall be compacted to a minimum of 95% of maximum density as determined by AASHO T-99, Method D, and AASHO T-147. Optimum moisture content shall be maintained within plus or minus 15% as determined by AASHO T-99, Method D. Gradation of material shall be as follows:

Sieve	Passing
1 1/2 inch	100%
3/4 inch	67- 82%
3/8 inch	37- 55%
# 10	13- 27%
# 40	7- 18%
#200.	4- 12%

Care was taken to obtain proper compaction at the haunches as illustrated in Figure 15. The backfill material was brought up equally on each side of the culvert and was compacted by small, pneumatic-tired rollers as indicated in Figure 16. This material was placed to a depth of five feet above the culvert and 12 feet on each side as indicated in Figure 2.

In late November and early December the straw was placed on the backfill material to form the imperfect trench. The straw was placed in two layers within the limits indicated in Figure 2 and as shown in Figure 17. The binder twine was cut after each layer of straw was placed. Loose rock fill was then placed on the straw and not compacted nor was any equipment allowed to cross this section until the loose material reached a height of eight feet above the straw. The remaining fill, consisting of material that was dumped by various hauling equipment, was then pushed into place



Figure 14. Walkways, Crosswalks and Folding Tables in Culvert



Figure 15. Compacting Material under Haunch



Figure 16. Backfill Material Placed in Six Inch Layers
(Looking Upstream)



Figure 17. Placement of Baled Straw
Five Feet above Culvert

and continually worked by a D-9 Caterpillar.

The remainder of the rock fill was completed early in January of 1966 and a gravel surface was placed as a temporary measure to open the highway to traffic. The final surfacing of asphaltic concrete, was placed in August of 1966.

Flexing of the plates resulting from placement of the fill relieved the tension in many of the bolts to the extent that the nuts could be turned with the fingers. In the summer of 1966 a work crew systematically tightened the accessible loose nuts.

Strain Gage Installation

In August of 1965, while stream diversion attempts were still under way, the culvert contractor placed six of the new 3/8 inch thick corrugated structural plates in an easily accessible outdoor work area. It was in this area that the SR-4 strain gages were applied, waterproofed, and checked before the plates were installed in the culvert.

Care was taken to prepare the surface of the plate where the gages were to be cemented to assure a good bond. Because of the rough exterior of the galvanizing at some locations, the area was filed and then finished with fine emery paper. In all cases the fine emery paper was used to assure a clean, lightly roughened surface to provide good adhesion. After this preparation, the surface was thoroughly cleaned with acetone. The

gages were then attached according to the directions included in the BLH Eastman 910 cement kit.

Gages were first placed on that side of the plates which would be on the inside of the culvert, and these gages were from a single lot having a gage factor of 2.13. A few of the gages were ruined during application or failed to meet the functional operation test and had to be replaced. More gages were required and the job was finished with gages from a second lot which had a gage factor of 2.08. Most of the gages that were placed on the side of the plate which would be on the outside of the culvert, were from the second lot. One of the disadvantages of this arrangement was that two compensating gages, one with a gage factor of 2.13 and one with a gage factor of 2.08, had to be used with five of the six instrumented plates.

Lead wires were attached using a rosin-core solder and a small electric soldering iron. Care was taken to eliminate any external strains by providing slack and taping the leads down. The gages and leads were thoroughly cleaned again with acetone, primarily to remove solder fluxing material which might have grounded or short-circuited the gages. The functional behavior of the gages was then checked according to the manufacturers recommendations after which the gages were water-proofed and protected by two layers of epoxy cement and a layer of silicone rubber

sealant. Each layer was allowed to dry before another was applied.

Figure 18 shows a plate with the strain gages attached and waterproofed.

Small holes were drilled through the plate so that the lead wires from those gages that would be on the outside of the culvert could be threaded to the inside of the culvert. The leads were protected by silicone rubber sealant as shown in Figure 18. These wires were then taped down on the inside so that they would be out of the way during erection.

The contractor placed the plates containing strain gages in pre-selected locations. One of these plates is illustrated in Figure 19. The gages were positioned on the plates so that they would be located slightly above the mid-height of the culvert. After the plates were in place, prefabricated angle irons, capped at each end with small metal plates which fit the corrugation curvature, were placed over those gages that would be against the fill material to protect them. Epoxy cement was used to attach the angle irons to the plate as shown in Figure 20. In each case the bottom end of the capped angle iron was butted against a seam or a bolt. For the gages shown in Figure 19, holes were drilled several inches below the gages for the angle iron support bolts.

Compensating gages with the same gage factor as the measuring gages were placed in the culvert near the measuring gages as indicated in Figure 21. These compensating gages were suspended by wire hangers from

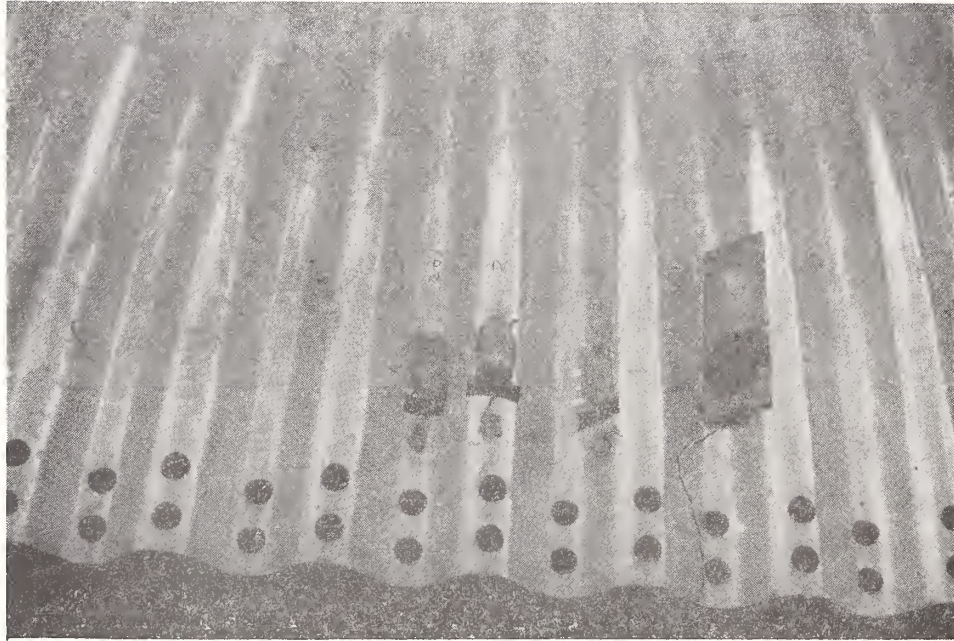


Figure 18. Mounted Strain Gages on Plate before
Erection and Compensating Gage

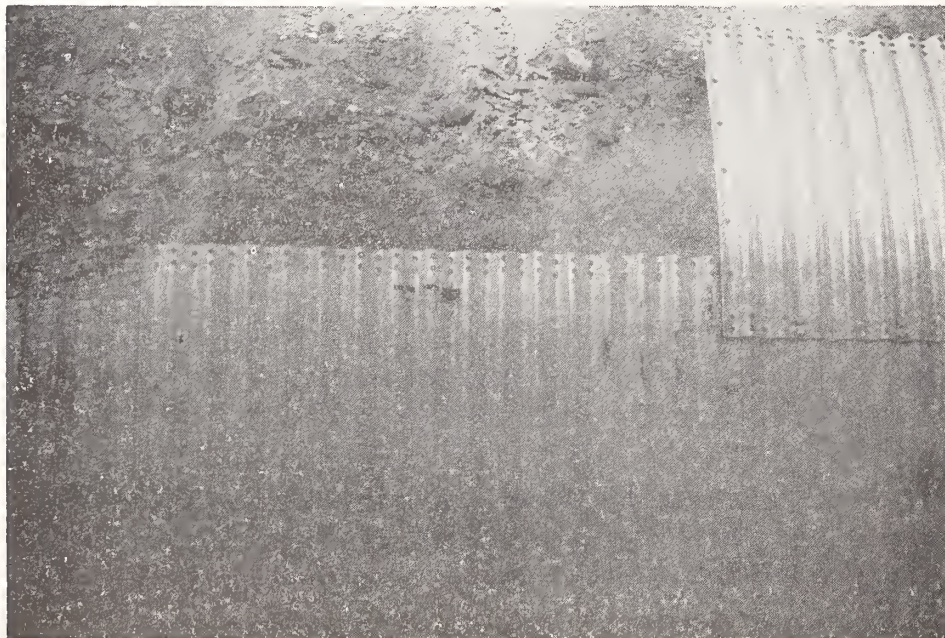


Figure 19. Three Strain Gages on Outside of Plate after Erection
(Located Near Top of Plate in Center of Picture)



Figure 20. Angle Iron Protection for Gages Against the Fill

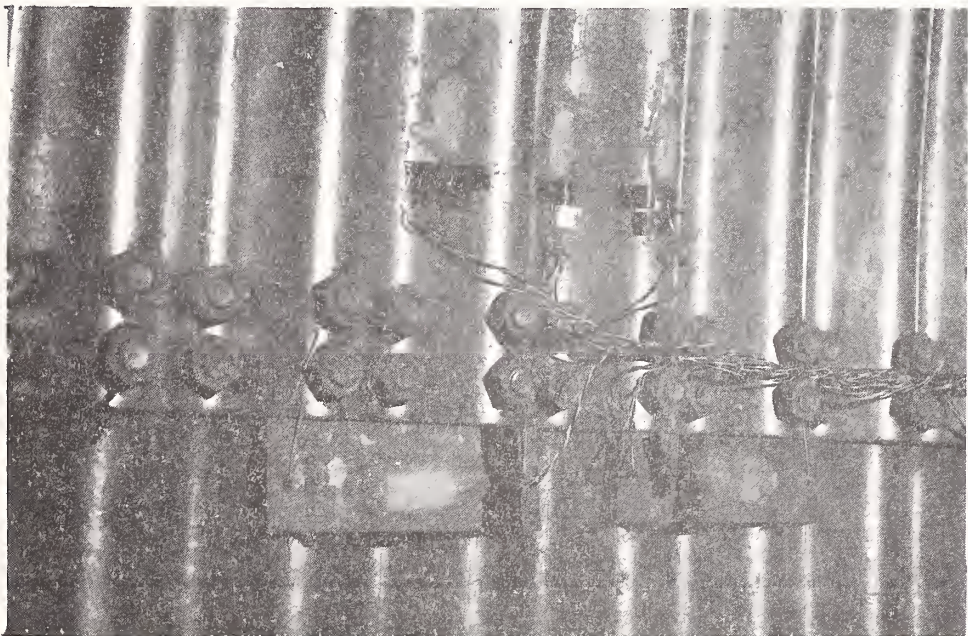


Figure 21. Compensating Gages Hanging in Culvert
Below Measuring Gages

erection bolts, so that external forces could not affect them. Spade type end connections were soldered to the lead wires of all gages and the wires were labeled, bundled and placed so that they would be accessible for taking readings.

The plates containing gages were identified as plates "A", "B", "C", "D", "E", and "F". The strain gages located on the inside of the culvert, using plate "A" as an example, were numbered "A1", "A2", and "A3" while those on the outside were "A11", "A22", and "A33" as shown in Figure 4. Figure 22 shows the locations of the plates on which strain gages were attached. The rings were numbered from 1 through 70 from downstream to upstream end, as they were in the original culvert, for identification purposes. Ring number 39 is under the center line of the Interstate roadway.

Pressure Cell Installation

Installation of the stress meters was started as soon as the necessary plates were erected by the culvert contractor. The holes for bolts were drilled and the holes for the cell stems were cut with an acetylene torch. All holes were painted with the same zinc oxide paint the contractor used for painting the reamed bolt holes on re-used plates.

Eighteen mounting frames were bolted to the culvert. In every case the bottom edge of the frame was butted against a bolted seam, as indicated

Figure 22., Location of SR-4 Strain Gages, Settlement Cells and Control Devices

in Figure 23. To provide a relatively rigid backing, the void area between the frame and the corrugated surface was filled with plaster of Paris or a sand-cement mortar as shown in Figure 24. Mounting frames were not used for those meters placed on the top center line of the structure. These meters were placed in a bedding of sand-cement mortar.

Lead cables were attached to the meters before placing them in the frames to assure good connections. After the meters were placed in the frames, the cables were strung to a switchboard inside the culvert so that all readings could be taken at one location. Three-pole, double-throw, knife switches, with large contact surfaces, were used to rapidly switch from one meter to another. The lengths of all cables were measured so that this additional resistance could be considered. One of the folding tables was located at this switchboard to facilitate the taking of readings. The switchboard, folding table and test sets for the stress meters and SR-4 strain gages are shown in Figure 25.

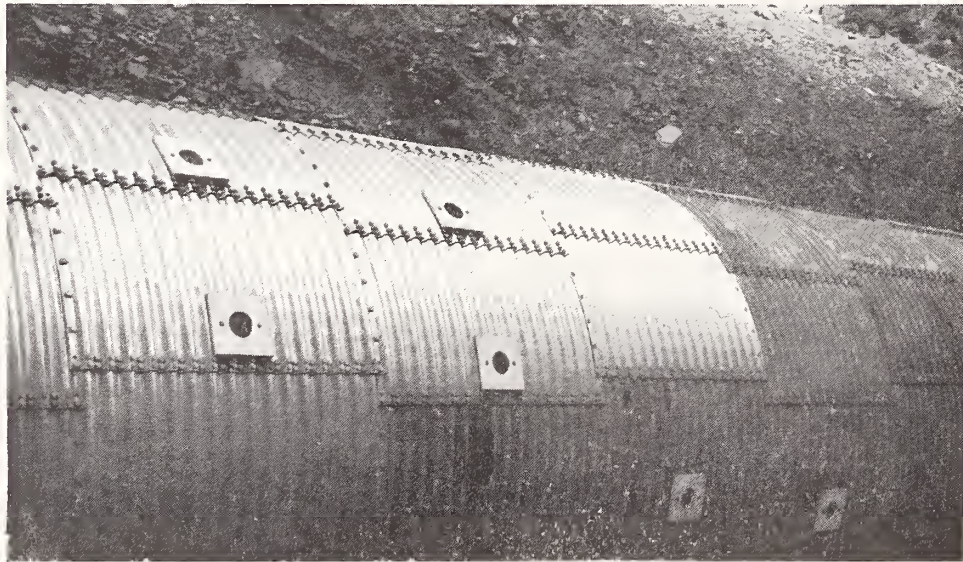


Figure 23. Mounting Frames for Stress Meters Installed on Culvert (Notice all Frames Butt against Bolts at all Seams)

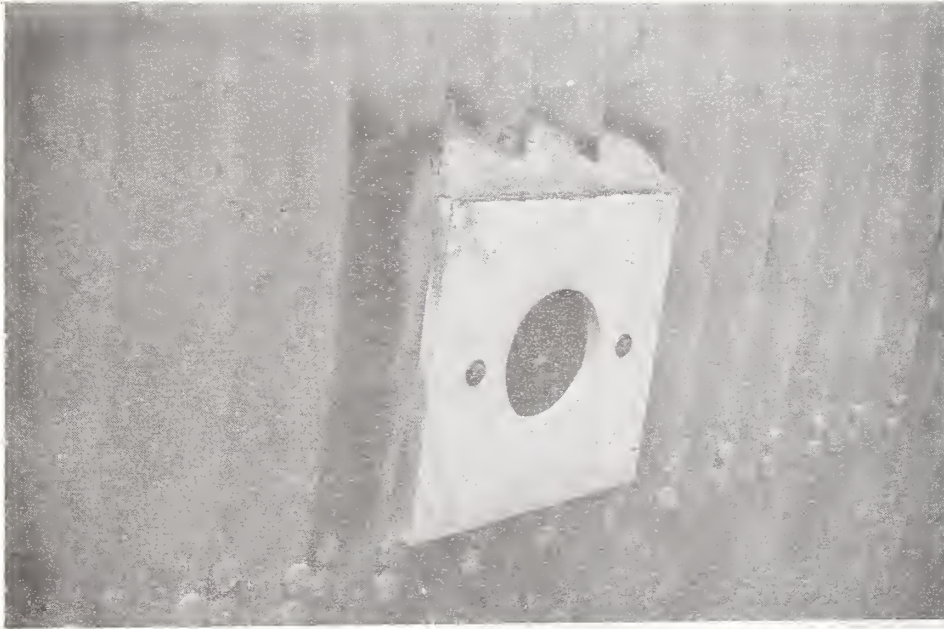


Figure 24. Mounting Frame with Mortar Backing

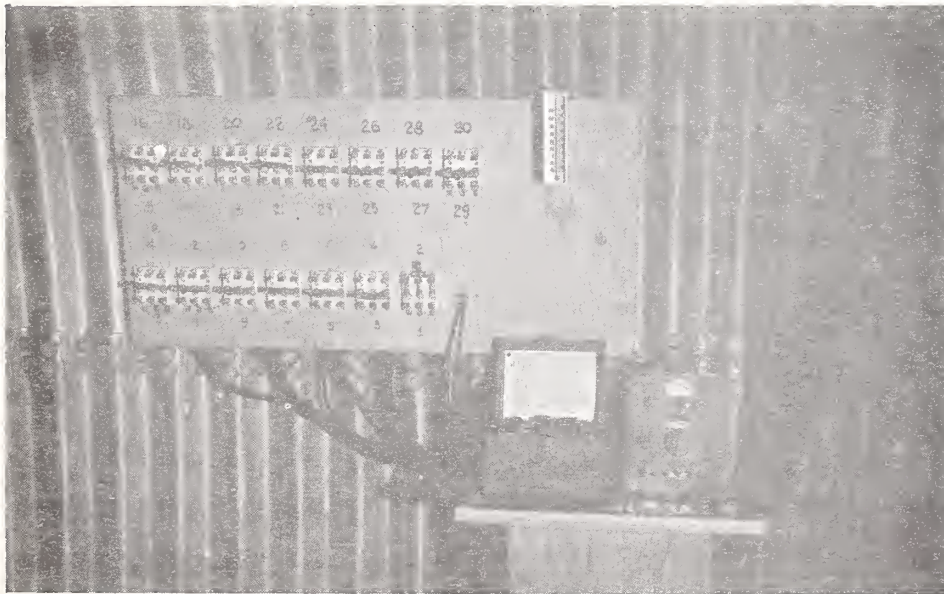


Figure 25. Stress Meter Test Set and Switchboard

Sand was placed against the face of the stress meters to keep the coarser material from damaging them or applying uneven pressure as the fill was placed. In spite of this precaution, construction equipment damaged some meters; note the tire marks on stress meter No. 17 shown in Figure 26. Stress meter No. 17 is also shown from the inside of the culvert in Figure 27 and shows clearly the protrusion at the back of the meter. Silicone rubber sealant was placed between the edge of the stress meter and the mounting frame to keep fine sand or backfill material from binding the edge of the meter.

Five stress meters were placed in the fill perpendicular to the culvert center line, as shown in Figure 28, above ring 43, when the backfill material was one foot above the culvert. The same type of mounting frame described earlier was used to protect those meters in the fill.

The location of the 30 stress meters and their identification numbers is indicated in Figure 29. From this it can be seen that the frame in the lower right corner of Figure 23 is for meter No. 8 and the frame in the upper left corner is for meter No. 18.

Six rubber pressure cells were placed perpendicular to the culvert center line, above ring 38, after the select backfill material was placed to its full height, five feet above the top of the culvert. In order to place the cells a trench was cut across the backfill down to the culvert.



Figure 26. Stress Meter Installed with Sand Protection
(Note Tire Mark)



Figure 27. Stress Meter from the Inside of the Culvert
(Note Stem of Stress Meter Protruding into
Culvert)

Holes were cut in the culvert with an acetylene torch for the copper tubing connecting the rubber bottles to the Bourdon gages.



Figure 28. Meters in Fill in Mounting Frames Perpendicular to the Culvert Center Line

Shelves were cut in the compacted backfill adjacent to the trench so that the rubber water bottles would be approximately six inches below the straw layer. The rubber bladders were placed on a sand bedding on the undisturbed backfill and protected by compacted sand as illustrated in Figure 30. The pressure cells were connected to Bourdon gages placed inside the culvert and the copper tubing was protected by sections of plastic water hose. A valve was placed in the line adjacent to the Bourdon gage as shown in Figure 31. Fluid was forced from the overfilled bottles to the valve in order to bleed the air out of the system. The location of the rubber pressure cells, with relation to the stress meters, is indicated in Figure 29.

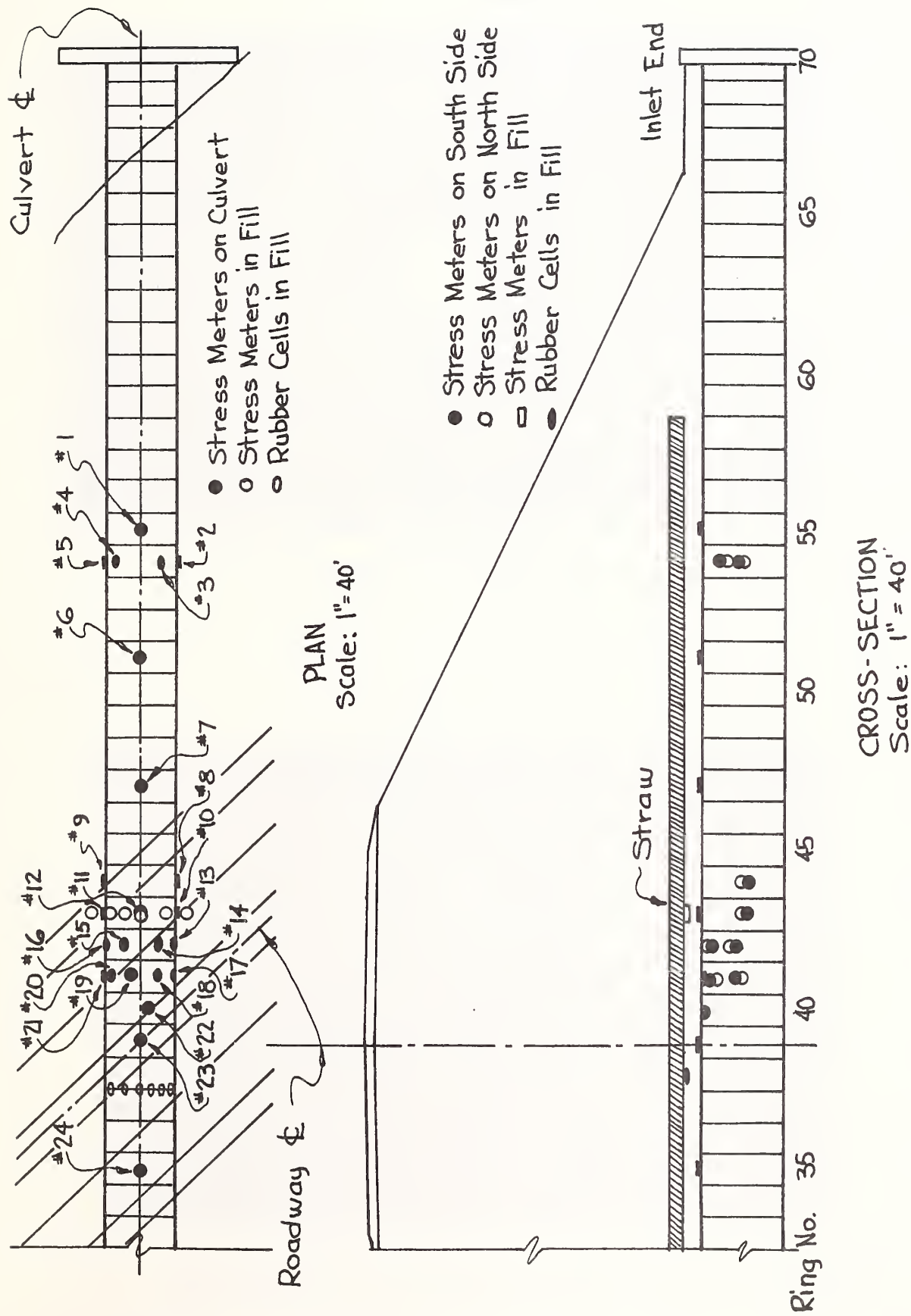


Figure 29. Location of Carlson Stress Meters and Rubber Pressure Cells

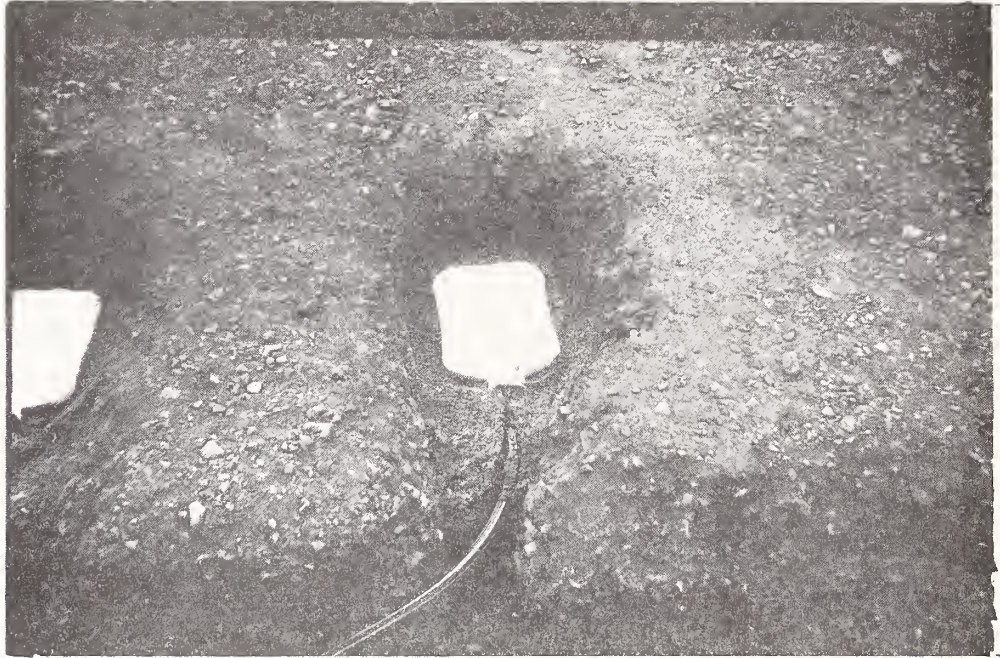


Figure 30. Rubber Pressure Cells Placed in Fill

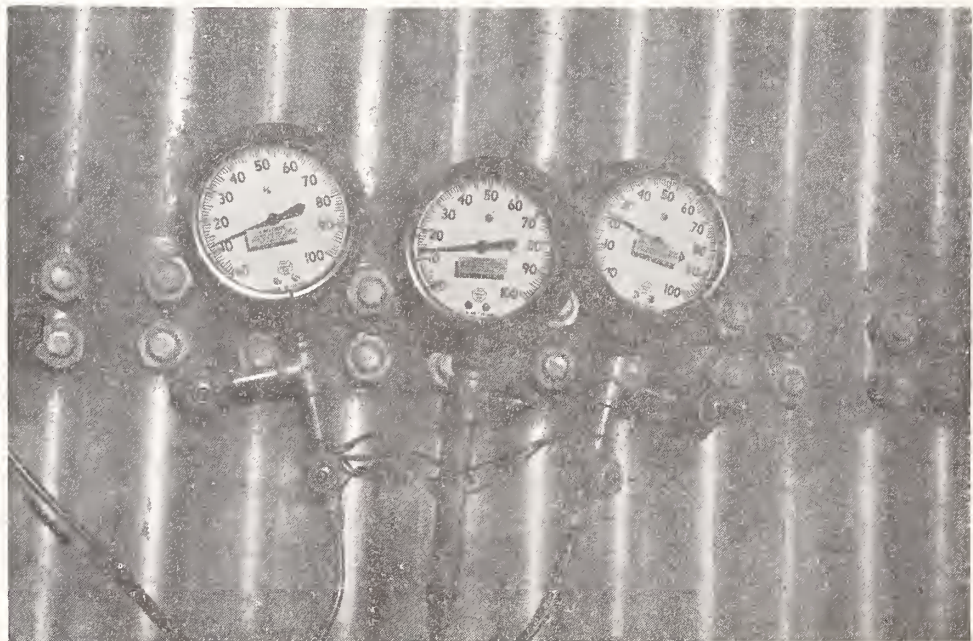


Figure 31. Bourdon Gages and Valves Inside Culvert

Settlement Cell Installation

Trenches were cut for the placement of the settlement cells at the same time that the trench was cut for the rubber pressure cells, when the backfill surface was five feet above the culvert. Cross-trenches were cut perpendicular to the culvert center line, 134 feet, 166 feet and 206 feet downstream from the culvert inlet. A trench was also cut along the south side of the culvert to the inlet for the 1/2 inch, black, iron pipe between the cells and the manometer board.

One cell was placed on the north side of each perpendicular trench and two on the south side. A settlement cell, in place, is shown in Figure 32. The location of the cells is shown in Figure 22. The cells were placed so that those on the south side were about a foot above the top of the culvert. The cells placed on the north side of the culvert were about a foot higher than those on the south side. This was to insure a gradient from the cell to the low point located below the manometer board.

The air and overflow lines were vented to the inside of the culvert through holes cut with an acetylene torch. Valves were placed at the low point of the main pipe line to facilitate filling and draining the system. The manometer board was attached to the concrete inlet headwall. The complete system at the headwall is illustrated in Figure 33.



Figure 32. Settlement Cell in
the fill



Figure 33. Manometer Board and Valves Mounted on Concrete
Headwall

CHAPTER VI

READING INSTRUMENTS AND TAKING MEASUREMENTS

Strain gage readings were taken as soon as the gages were attached to the plates and while the plates were still lying individually on the ground in the stockpile area. These initial readings established a basis for zero strain so that any additional strain could be determined.

After the culvert was erected and before any backfill was placed, readings were taken in order to determine the strains caused by erection. Readings were then taken at close time intervals until the full fill height was reached. The intervals between readings were increased after the fill was completed. Special readings were also taken periodically to check for indicator drift which proved to be negligible.

Stress meter readings were taken for all meters at the central switchboard. Preliminary readings were taken to determine the effect of the various lengths of cable connecting the meters to the switches and corrections were made for this added resistance. Stress meter readings were also taken before and after the switches were connected but there was no indication that the switches affected the readings. The readings consisted of readings for determining temperature corrections and readings for determining the pressure on each meter. Stress meter readings were taken at the same time that the strain gage readings were taken.

After the rubber pressure bladders were connected to the Bourdon gages and the air removed, the gage pointers were adjusted to read zero.

This gave an initial zero reading for measuring earth pressure corrected for the pressure caused by the difference in elevation between the Bourdon gage and the rubber cell. Readings of the Bourdon gages were taken at the same time as those of the strain gages and the stress meters.

Each settlement cell was made operational by filling the system under pressure with a garden spray pump which is visible in Figure 33. Initially, a half-and-half mixture of water and anti-freeze was used to prevent freezing. The pump was connected to the valve at the low point in the system and fluid was forced into the line, with the top of the standpipe plugged to prevent overflow, until the lines were completely filled. At this time the valve was closed, the pump disconnected and the plug at the top of the standpipe removed. After the fluid settled in the standpipe to the level of the cell, a plastic scale was fastened to the board with the zero mark at the fluid level. Settlement at any subsequent time was then read directly as the scale reading of the liquid surface.

A small amount of liquid was added, to eliminate possible leakage or evaporation errors, to each standpipe and allowed to stabilize before reading the settlement. After the first few weeks the readings became highly erratic. Part of this trouble was apparently due to the vent lines being intermittently plugged with frost. Also, temperature differences and differences in anti-freeze concentration affected the density of the

fluid in the manometer, as compared to that in the lines in the fill, enough to give incorrect readings. The readings were highly erratic for several months. Finally, proper functioning was restored by draining the system and keeping it empty between settlement observations. Readings were then taken by filling the system with water as previously explained, letting the system stabilize and, after recording the new settlement level, draining the system. Reproducibility was generally better than plus or minus 0.20 inches by this method.

Measurements of the horizontal and vertical culvert diameter were taken with a telescoping rod at 13 stations inside the culvert at the control devices, as soon as the culvert was erected and before any backfill was placed. Periodic re-measurements at the same stations during placement of the backfill provided a valuable record of culvert deflections that yielded information about the behavior of the reconstructed culvert. The control devices on the culvert floor were used as measuring stations for determining vertical diameter changes and floor settlement.

Plumb bobs were hung from the roof of the culvert directly above the rods in the control devices to determine any rotation of the culvert. No measurable rotation of the culvert was detected during placement of the fill or thereafter. Observations for rotation were discontinued in

February of 1966 when ice and high water bent the rods. The rods were straightened after high water receded and were again used for vertical diameter and floor settlement measurements.

Floor settlement was obtained from periodic observations of the elevation at the top of each control device rod, using an engineer's level near the culvert inlet. These observations were necessarily made during periods of low water.

Levels were taken periodically to determine if there was excessive settlement of the roadway surface above the culvert. These levels were started in September of 1966, as soon as the asphaltic concrete surfacing was completed.

CHAPTER VII

REDUCTION OF DATA

Data taken in the field had to be reduced to give information that would be of value in determining the behavior of the structure. The strain gage readings, in micro-inches per inch, were reduced to give the stresses acting on the plate and the equivalent uniform earth pressure that would cause those stresses. Bending moments in the plates were also calculated from the observed stresses. Stress meter readings were in units of resistance, ohms, and had to be converted to give temperatures and earth pressures. Readings produced by the rubber pressure cells and settlement cells were read directly in psi and did not have to be reduced. Physical measurements such as horizontal and vertical diameters and invert elevations were used directly, except when they were subtracted from a base measurement to show a change.

Strain Gages

Strain gage readings for zero reference purposes were taken while the plates were in the stockpile area prior to erection of the culvert. The strain readings taken immediately after erection of the culvert were subtracted from the zero reference readings to determine the erection strains and stresses. The difference between the reference reading and any reading taken while the fill was being constructed, or after construction was completed, gave the strain in the

culvert caused by erection and earth pressure. Also, the difference between the reading taken immediately after culvert erection and any later reading gave the strain in the culvert attributable to earth pressure only. The latter relationship is valid only when all stresses are below the proportional limit.

The strain gages, installed as shown in Figure 4, provided measurements of circumferential strain at the extreme fibers as well as the circumferential strain at the geometric neutral axis of the sidewall. When the strains were within the elastic range of the steel, the stress was determined as the product of the strain and the modulus of elasticity (30×10^6 psi).

The stress pattern resulting from the combined bending and compression was used to calculate both the bending moment and compressive force. The typical stress diagram of Figure 4 will be used to illustrate these calculations. Assuming a linear relationship in the elastic range, the method of least squares was used to determine the stress line between extreme fibers of the plate. The combined stress at any distance, x , from the neutral axis may be expressed as:

$$S_C = \frac{P}{A} \pm \frac{Mx}{I}$$

Where:

S_C = combined stress, psi

P = total compressive force, pounds per
inch of culvert length

A = cross-sectional area of the culvert wall, in^2
per inch of length
= 0.467 in^2 per inch for the $3/8$ inch plate
of the Wolf Creek culvert

M = bending moment, inch-pounds per inch of length

I = moment of inertia of the wall cross-section,
 in^4 per inch of length
= 0.234 in^4 per inch of the Wolf Creek culvert

$\frac{P}{A}$ = direct compressive stress, S_d , psi

$\frac{Mx}{I}$ = bending stress, S_b , psi

At the extreme fibers:

$x = c = 1.188$ inches for the Wolf Creek culvert

At the neutral axis, where x is equal to zero, the direct compressive stress is about 9000 psi in Figure 4. In this example the total compressive force is then:

$$P = S_d \cdot A = 9000 (0.467) = 4200 \text{ lb/in}$$

The bending stress is determined by algebraically subtracting the compressive stress from the combined stress at any point. At the extreme fibers, the bending stress has a numerical value of about 14,000 psi in the example of Figure 4. In the same example, the bending moment, in-lb per inch of culvert length, is then:

$$M = S_b \frac{I}{c} = \frac{14,000 (0.234)}{1.188} = 2760 \text{ in-lb/in}$$

Figure 34 shows a free body diagram of the top half of a section of

the culvert and shows how the compressive force acting in the culvert wall may be related to the average equivalent vertical earth pressure.

The total vertical load on the culvert can be expressed as

$$V = (w)(D)(L)$$

Where:

V = total vertical load, pounds, on a section L inches long

w = average equivalent vertical earth pressure, psi

D = culvert horizontal diameter, inches, taken as 216 inches for the deflected Wolf Creek culvert

L = length of section, conveniently taken as one inch,

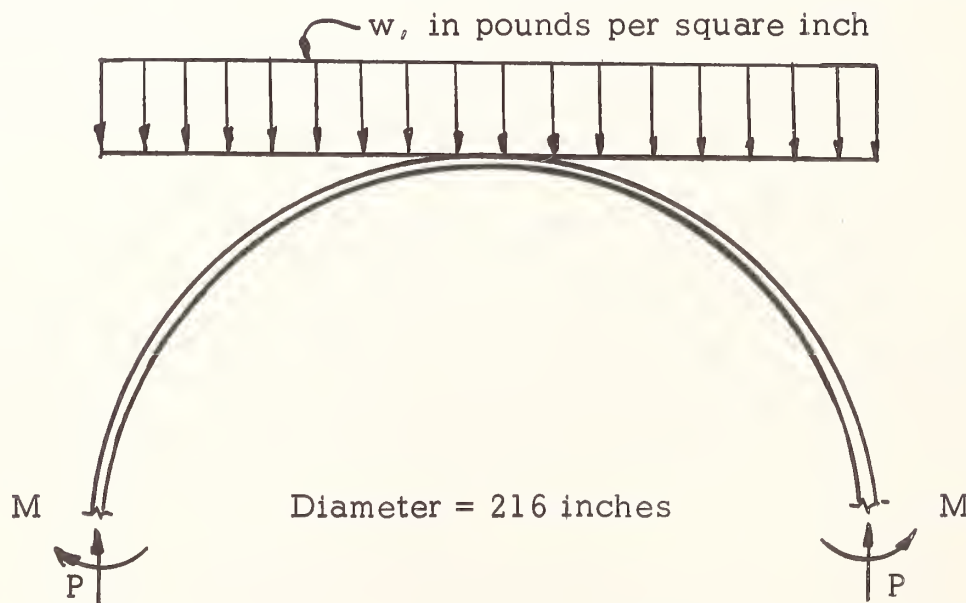


Figure 34. Determining Earth Pressure from Compressive Force in Walls

It is evident, from the free body diagram, that the total vertical load, V , is also equal to $2P$; therefore:

$$V = 2P = wDL \text{ or } w = \frac{2P}{DL}$$

These equations were used to compute the average earth pressure, w , corresponding to the vertical compressive force, P .

In some cases the elastic limit was exceeded, and in these cases a numerical summation process was used to calculate the compressive force and bending moment. For this condition the slope of the stress line was calculated by the same method as before except that the stress increase was terminated at the yield stress of 30,000 psi. The compression force and bending moment was found as illustrated in Figure 35. The compression force, P , was determined by taking the algebraical summation of the area of the segments of the plate times the average stress acting on the segment:

$$P = \sum (S) (A')$$

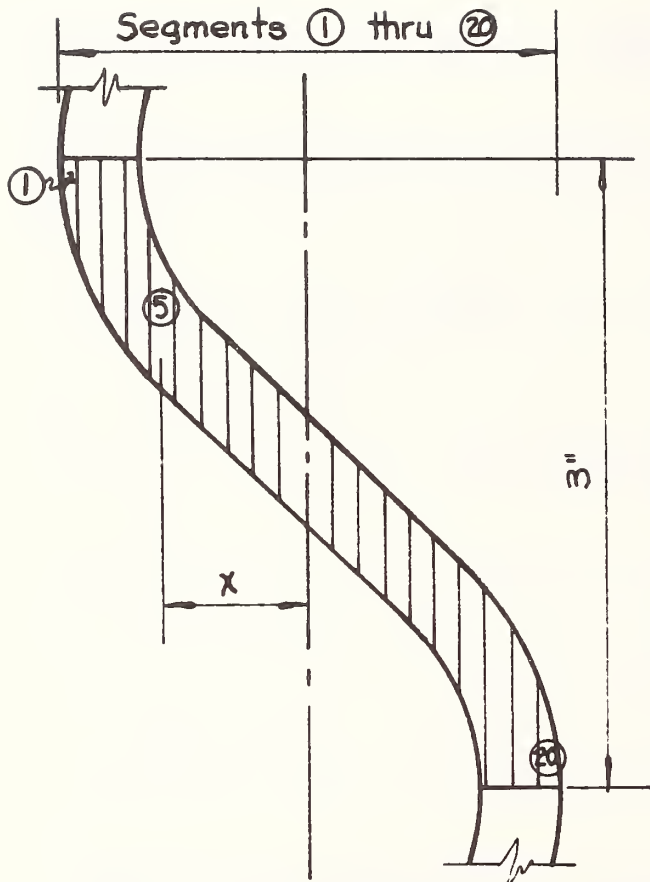
Where:

S = stress, psi, acting on A'

A' = area of segment, in²

The area was calculated for 10 segments, of equal width, on each side of the neutral axis or a total of 20 segments as shown in Figure 35. The bending moment, M , was determined by taking the summation of the products calculated above times the distance between the area and the geometric neutral axis:

$$M = \sum (S_c) (A') (x)$$



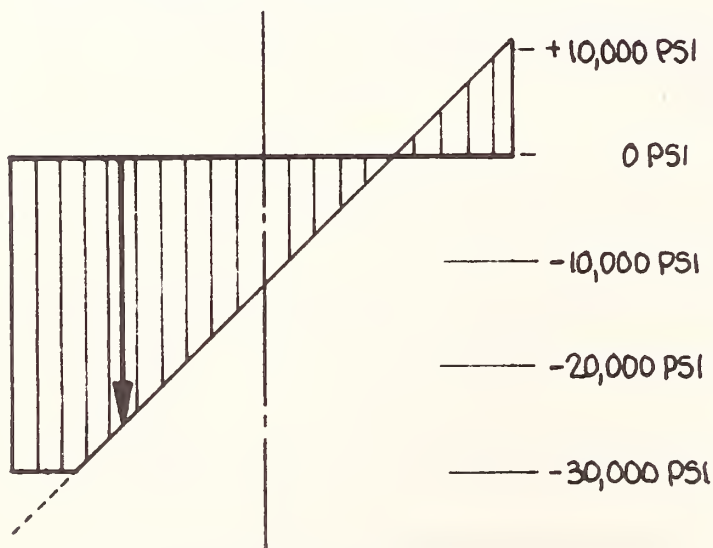
Force on Segment ⑤
= Stress at ⑤ times Area of ⑤

$P = \text{Total Force} = \sum \text{Stress} \cdot \text{Area}$
Earth Pressure, $w = \frac{2P}{DL}$ as in Fig. 34

Moment from Segment ⑤ Force
= Force on ⑤ times x

Total Moment = $\sum \text{Force} \cdot x$

SECTION OF $\frac{3}{8}$ INCH PLATE



STRESS DIAGRAM

In Elastic Range Only:

Stress = $E \cdot \text{Strain}$
 $E = 30 \times 10^6 \text{ PSI}$

Yield Stress = 30,000 PSI

Figure 35. Earth Pressure from Inelastic Strain

Where:

x = distance between area and neutral axis, inches

With the compression force determined, the average earth pressure was computed as indicated by Figure 34. The yield stress of 30,000 psi was established from two tensile tests and may not be representative of the total structure. A different yield stress, or even ignoring the yield, would not change the calculated average earth pressure to any great extent.

Because of the large number of readings taken, a computer program was developed to perform the computations. The program also printed out identification information such as the plate and gage numbers, the date, the height of cover and the stress at each gage. Computations included; the slope of the best line to fit the stress coordinates, the stress at the extreme fibers, whether or not the calculated stress at the extreme fibers exceeded the yield stress, the uniform average earth pressure that would cause that particular stress condition, and the bending moment in the wall at the strain gage location. The computer program is given in Appendix A.

Stress Meters

The stress meter readings had to be reduced to obtain the meter temperature and the pressure acting on the meter at the time of each

reading. The calibration information received from the manufacturer and checked at the University was used, with slight modifications, to convert the meter readings into temperatures and earth pressures. Figure 36 illustrates how earth pressure was determined from the readings of stress meter No. 1. The two charts were constructed from the basic calibration data for meter No. 1, and their use is illustrated by the following numerical example: Assume that the temperature reading R_t is 74.4 ohms at the same time that the raw resistance reading RR_p is 104.18 ohms. Entering the graph with the temperature reading of 74.40 ohms it is found that the temperature is approximately 60 degrees Fahrenheit and the correction of the resistance reading is plus 0.02 ohms. Applying the correction to the resistance reading and entering the graph on the left with 104.20 ohms the earth pressure is found to be 25.5 psi. This method was used to reduce the stress meter readings to earth pressures and a separate graph had to be prepared for each meter.

The tedious graphical reduction of the stress meter readings was abandoned after a computer program was developed to perform the same task analytically. This program printed out the date the reading was taken, the height of cover on that date, the meter identification number, the temperature at the time of reading, and the earth pressure. The computer program is given in Appendix B.

Standard Conditions, Stress Meter No. 1:
 Resistance Reading at Zero Pressure and 42°F., $RR_0 = 104.61$ Ohms
 Resistance Reading Correction = $0.606 \text{ PSI} / 0.01 \text{ Ohm}$
 Reading at Zero Temperature, $R_0 = 67.07$ Ohms
 Temperature at Zero Correction = 42.0°F
 Temperature Factor = $7.89^\circ \text{ Ohm for } R_T$

Example:

$R_T = 74.4$ Ohms $RR_P = 104.18$ Ohms
 Enter R_T and Find RR Correction
 Correction = $+0.02$ and Temp = 60° F
 Corrected $RR_P = 104.18 + 0.02 = 104.20$ Ohms
 Enter Corrected RR_P and Find Pressure
 Earth Pressure = 25.5 PSI

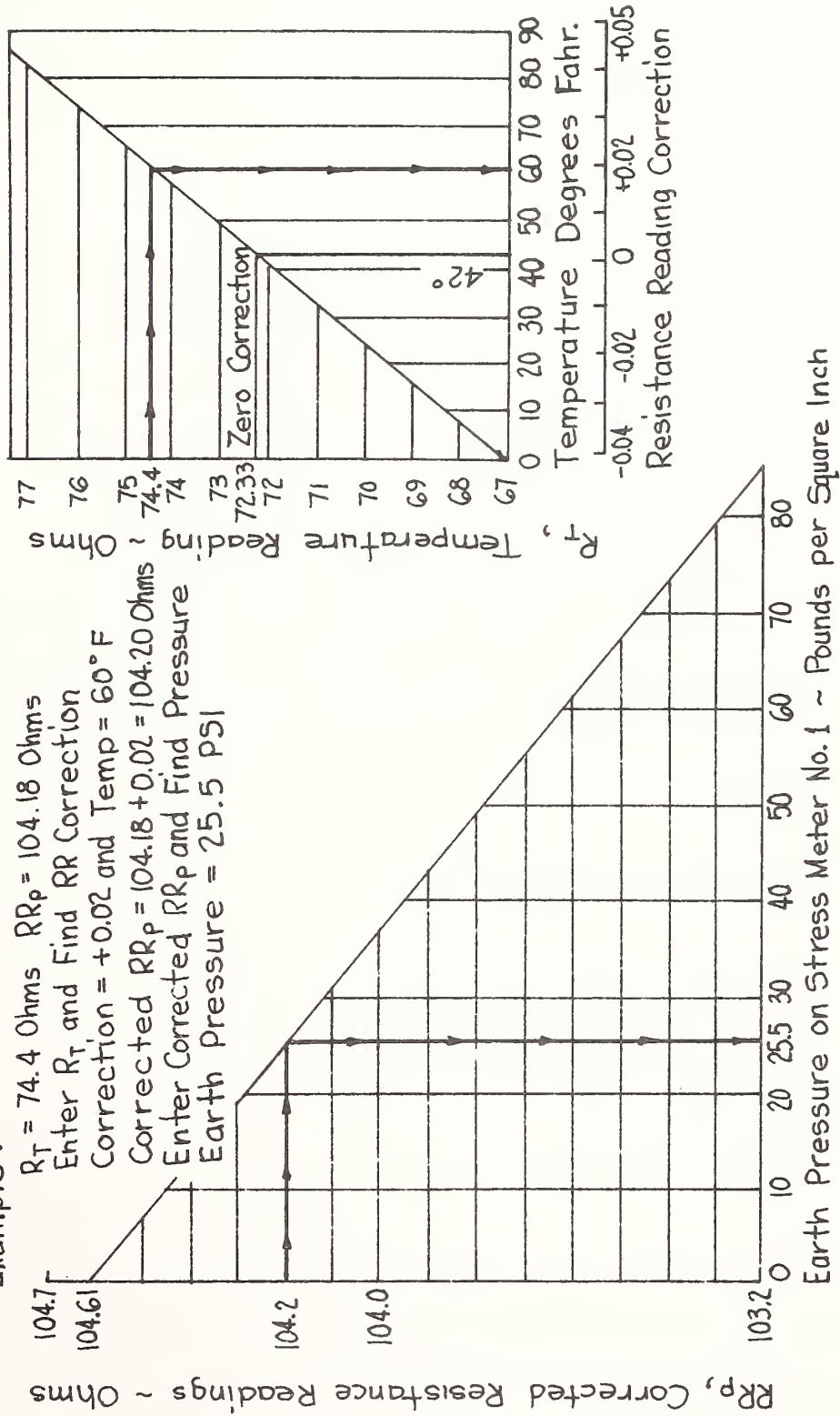


Figure 36. Determining Temperature and Earth Pressure from Stress Meter Readings

CHAPTER VIII

RESULTS AND ANALYSIS OF DATA

Data was compiled from the various sets of instrumentation during each phase of construction and analyzed to determine the behavior of the culvert.

Strain Gages

Strain gage measurements were taken before and immediately after the culvert was erected in order to determine the circumferential erection stresses at the strain gage sites. The extreme fiber erection stresses ranged from 3,800 psi in plate C to 21,900 psi in plate A. Plates B, D, and E might be considered typical, with erection stresses between 9,000 and 10,000 psi in the extreme fibers. Plate A was one of the last plates to be installed and had to be forced vigorously into place which may account for its high stress of 21,900 psi. All of the gages were operational when the erection stresses were determined except for gage D22 which was destroyed by a bulldozer during backfilling operations and gage C22 which gave erratic readings and may have been damaged also.

The erection stresses are shown graphically in Figure 37. The excellent linearity of the stress diagrams, from the extreme outside fibers to the extreme inside fibers of each plate, shows that the gages performed in a highly consistent fashion. This consistency is an indication of reliable results. Also, the very low stresses midway between the

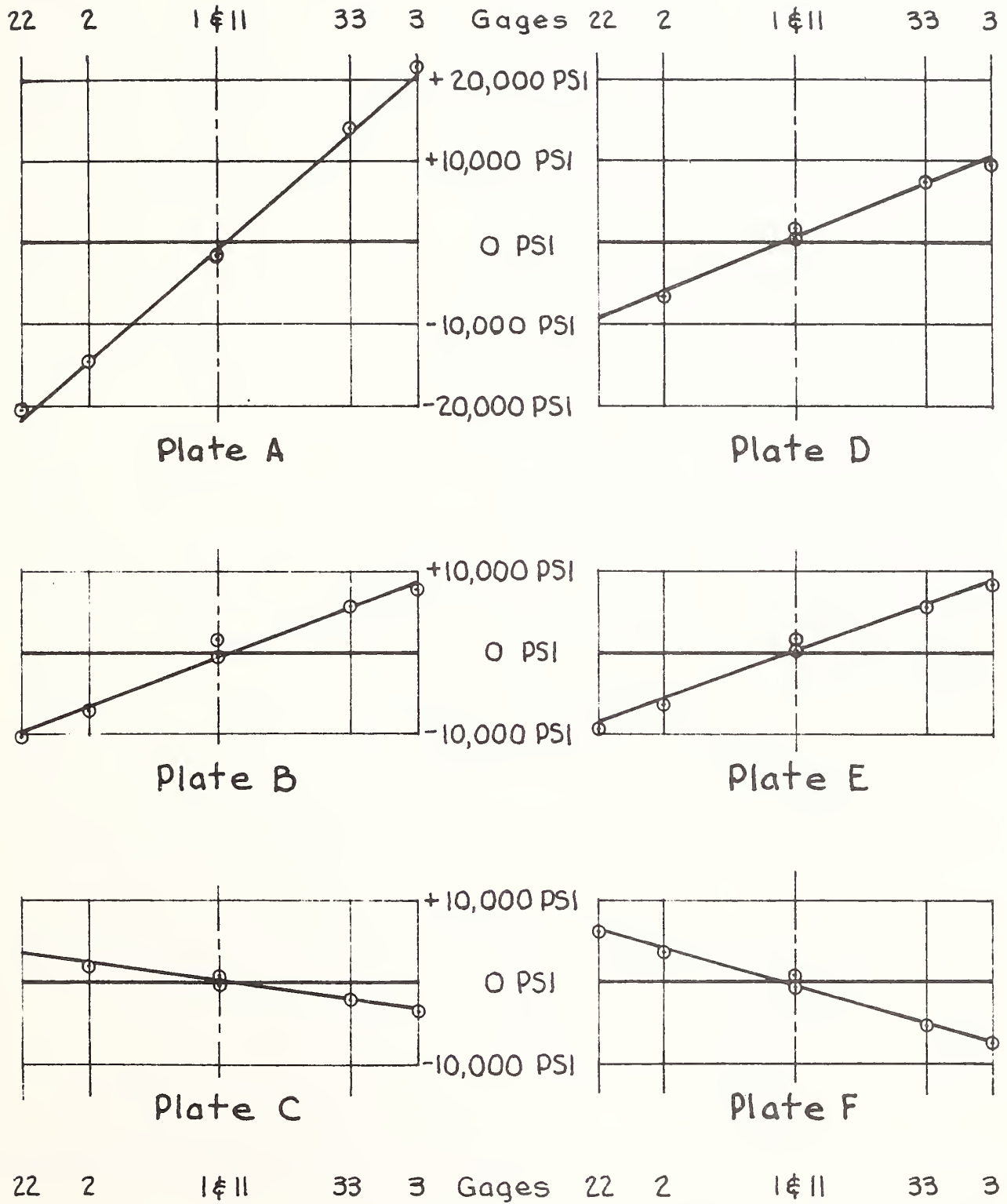


Figure 37. Erection Stress in the Six Instrumented Plates

extreme fibers show that the erection stresses were largely the result of bending of the plates during erection, which is not surprising. Some bending is often required to insert the bolts, and additional bending takes place when the nuts are tightened. The small amount of axial erection stress, P/A , that is apparent in Figure 37, is not surprising either. It could easily result from the pushing or pulling required to make some plates fit.

Table 2 summarizes the erection stresses, which were segregated into axial and bending components as previously described, and also shows the bending moments required to produce the observed erection stress patterns.

TABLE 2. STRESSES CAUSED BY ERECTION

Plate	Combined Stress psi	P/A Stress* psi	MC/I Stress psi	Bending Moment to cause MC/I** In-lb/in
A	21,900	- 800	21,100	-4,200
B	9,600	-1,000	8,600	-1,700
C	3,800	- 400	3,400	+ 700
D	9,600	- 4	9,600	-1,900
E	9,200	- 400	8,800	-1,700
F	7,800	-1,100	6,700	+1,300

* Compression -
Tension +

** Outside fibers (adjacent to backfill) in compression-
Outside fibers (adjacent to backfill) in tension +

Strain gage measurements were taken at intervals as the backfill was placed to a level 5 feet above the culvert and as the rock fill was placed to a depth of 83 feet above the culvert. Stresses caused by these operations are shown in Figure 38 and 39, and pertinent results are summarized in Tables 3, 4, 5, and 6. While the backfill was being placed and compacted on each side of the culvert, large changes in bending stress occurred but there was little increase in vertical axial compression stress as would be expected. The combined stress at the extreme fibers of plate A reached the yield stress of approximately 30,000 psi before the depth of cover reached 5 feet.

The placement and compaction of the backfill on the sides of the culvert caused a reduction of approximately 2 inches in the horizontal diameter and a corresponding increase in the vertical diameter, and produced a compressive bending stress in the outside fibers and tensile bending stress in the inside fibers of the sidewalls, at the strain gage sites. (All gages number "22" were on the extreme outside fibers and all gages numbered "3" were on the extreme inside fibers of the sidewalls).

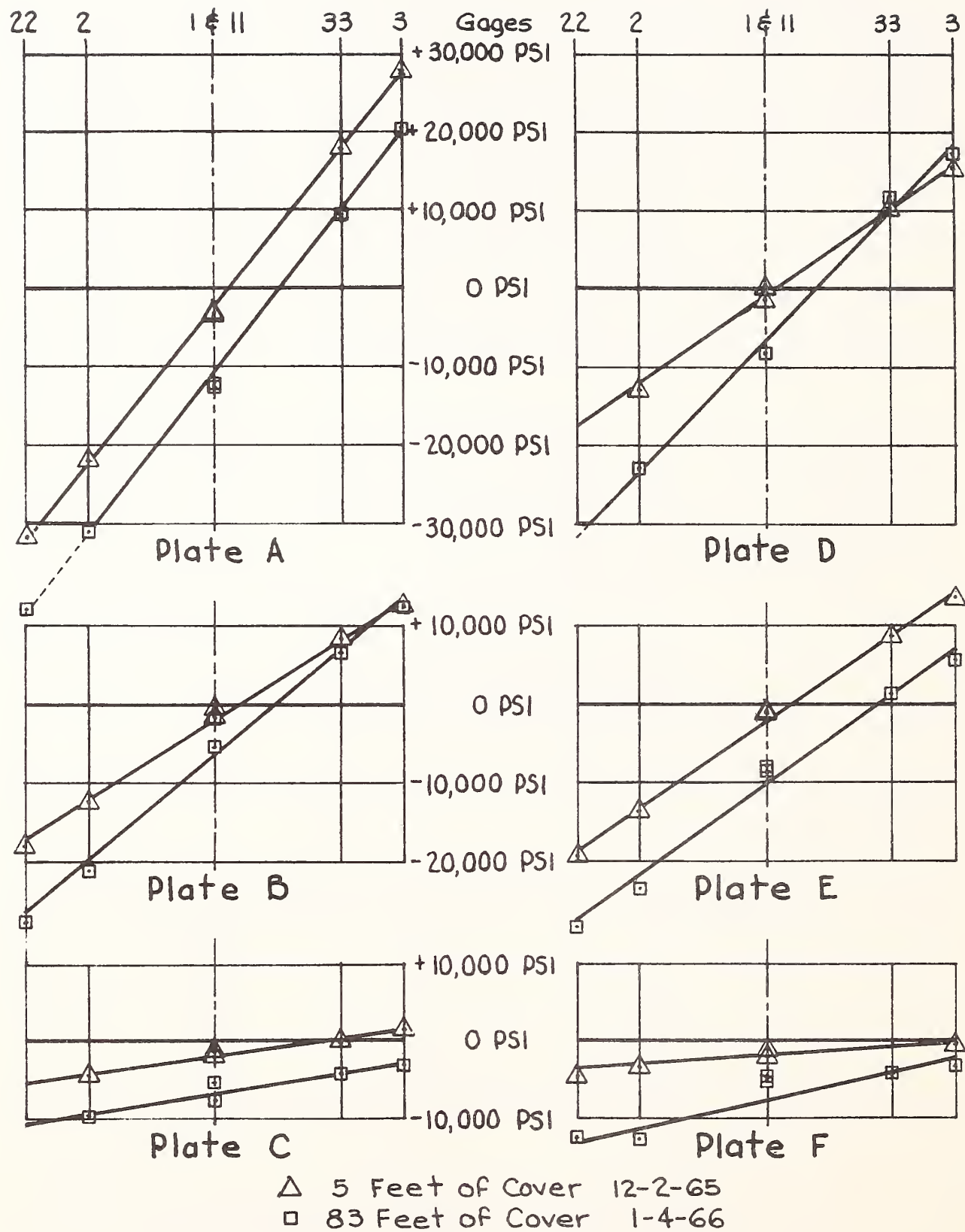


Figure 38. Stresses Caused by Erection and Earth Pressure

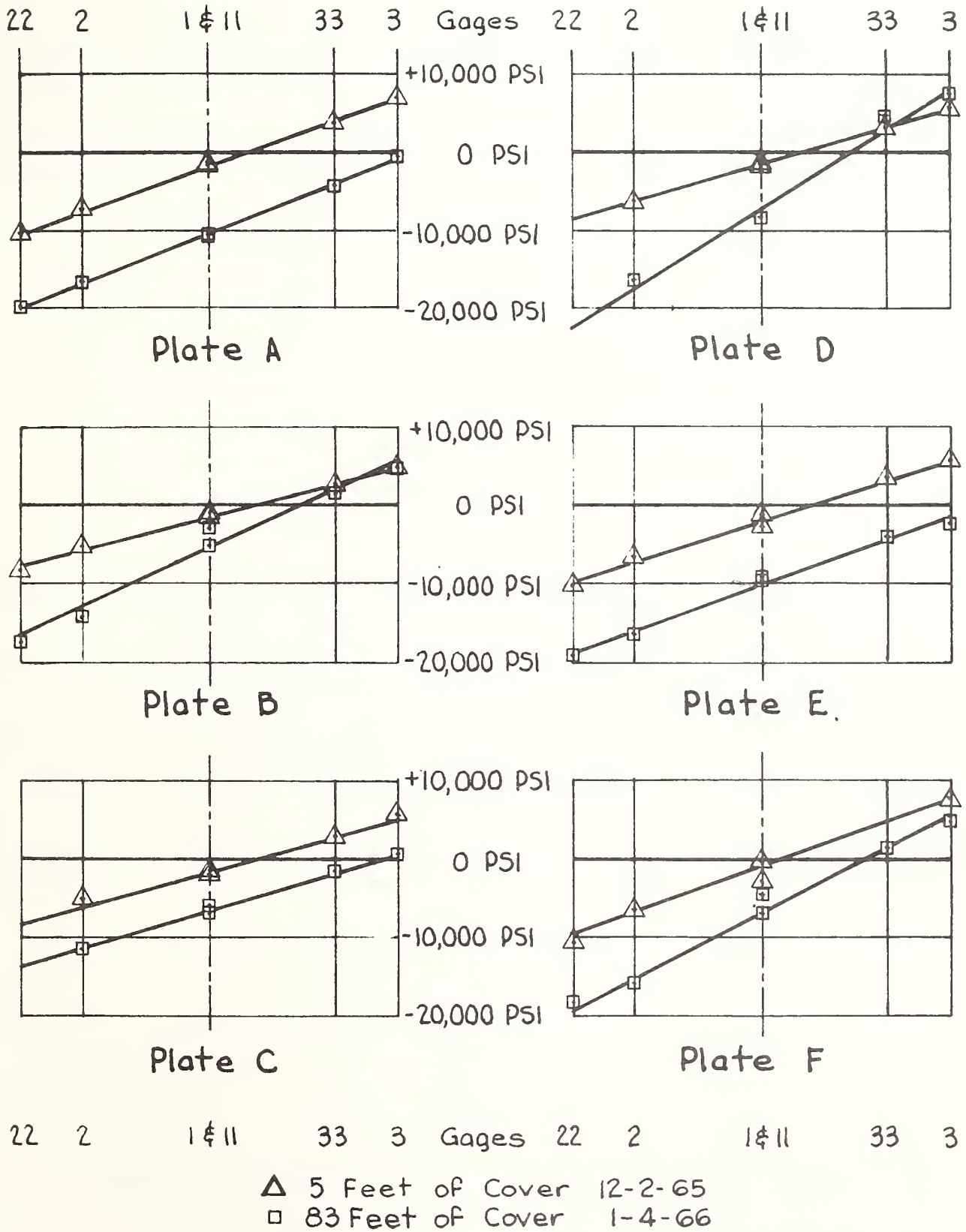


Figure 39. Stresses Caused by Earth Pressure

TABLE 3. STRESSES CAUSED BY ERECTION AND FIVE FEET OF COVER - 12/2/65

Plate	Combined Stress psi	P/A Stress* psi	MC/I Stress psi	Bending Moment to Cause MC/I** In-lb/in
A	30,000***	-2,500	27,500	-5,800
B	17,000	-2,200	14,800	-2,900
C	5,500	-2,000	3,500	-700
D	18,300	-1,500	16,800	-3,300
E	18,400	-2,200	16,200	-3,200
F	3,600	-1,800	1,800	-400

TABLE 4. STRESSES CAUSED BY FIVE FEET OF COVER - 12/2/65

Plate	Combined Stress psi	P/A Stress* psi	MC/I Stress psi	Earth Pressure to Cause P/A psi	Bending Moment to Cause MC/I** in-lb/in
A	8,100	-1,700	6,400	7	-1,700
B	7,400	-1,200	6,200	6	-1,200
C	8,500	-1,600	6,900	7	-1,400
D	8,700	-1,500	7,200	7	-1,400
E	9,200	-1,800	7,400	8	-1,500
F	9,100	-700	8,400	3	-1,700

* ** See Footnotes on page 68

*** Yield stress

TABLE 5. STRESSES CAUSED BY ERECTION AND 83 FEET OF COVER -
1/4/66

Plate	Combined Stress psi	P/A Stress * psi	MC/I Stress psi	Bending Moment to Cause MC/I** in-lb/in
A	30,000***	-9,800	****	****
B	26,800	-7,000	19,800	-3,900
C	11,300	-7,300	4,000	-800
D	30,000***	-7,200	22,800	-4,700
E	28,000	-11,000	17,000	-3,300
F	12,900	-7,600	5,300	-1,000

TABLE 6. STRESSES CAUSED BY 83 FEET OF COVER - 1/4/66

Plate	Combined Stress psi	P/A Stress* psi	MC/I Stress psi	Earth Pressure to Cause P/A psi	Bending Moment to Cause MC/I** in-lb/in
A	****	-9,000	****	39	****
B	17,100	-6,000	11,100	26	-2,200
C	14,300	-6,900	7,400	30	-1,500
D	21,500	-7,200	14,300	31	-2,800
E	18,800	-10,500	8,300	45	-1,600
F	18,400	-6,500	11,900	28	-2,400

* See Footnotes on page 68 and 72.

**** Not determined because of extent of yielding.

For plates A, B, D, and E, these bending stresses were directly additive to erection stresses and therefore produced relatively large compressive stresses in the outside fibers. For plates C and F, however, these stresses were of opposite sign to the erection stresses and therefore had a cancelling effect which explains the flat slopes and low stresses of the stress diagrams for plates C and F in Figure 38.

The rock fill was placed in a one and one-half month period between the middle of November of 1965 and the first of January, 1966. The stresses on January 4, 1966, just a few days after completion of the fill, are shown in Figures 38 and 39 and Tables 5 and 6. During construction of the rock fill, the combined stress in plate D reached the yield point in the extreme outside fibers. This is evident in Figure 38 which also shows that inelastic action occurred in plates A and D only, of the instrumented plates, and was limited to rather small zones in each of those two plates. The small extent to which inelastic action was present was not considered sufficient to seriously invalidate the superposition technique that was used to obtain the stresses caused by the embankment load only, in Figure 39 and Tables 4 and 6. By this technique, erection stresses were subtracted from total stresses to get the stresses attributable to the embankment load. To put it somewhat differently, these stresses are those that

would have been computed if the SR-4 gages had been cemented to the walls after the culvert was erected, or if readings had not been taken until after the culvert was erected.

The consistent slope pattern of the stress diagrams in Figures 38 and 39 show that there was little basic change in the bending stress patterns at the strain gage sites as the height of cover was increased from 5 feet to 83 feet. In every case the bending stress remained compression in the outside fibers. Undoubtedly the sidewalls would eventually have bulged outward sufficiently to reverse the early bending pattern if the adjacent backfill had been poorly compacted. As it turned out, the sidewalls never regained the original position that they had before the placement and compaction of the backfill material.

Figure 40 indicates the stresses caused by the additional 78 feet of cover over the 5 feet indicated in Figure 39. The almost level lines for plates A, C, and E demonstrate that the additional earth load caused compressive stresses primarily. In plates B, D, and F the bending stresses were significant. Table 7 contains the tabulation of these stresses along with the additional earth pressures and bending moments caused by this additional cover.

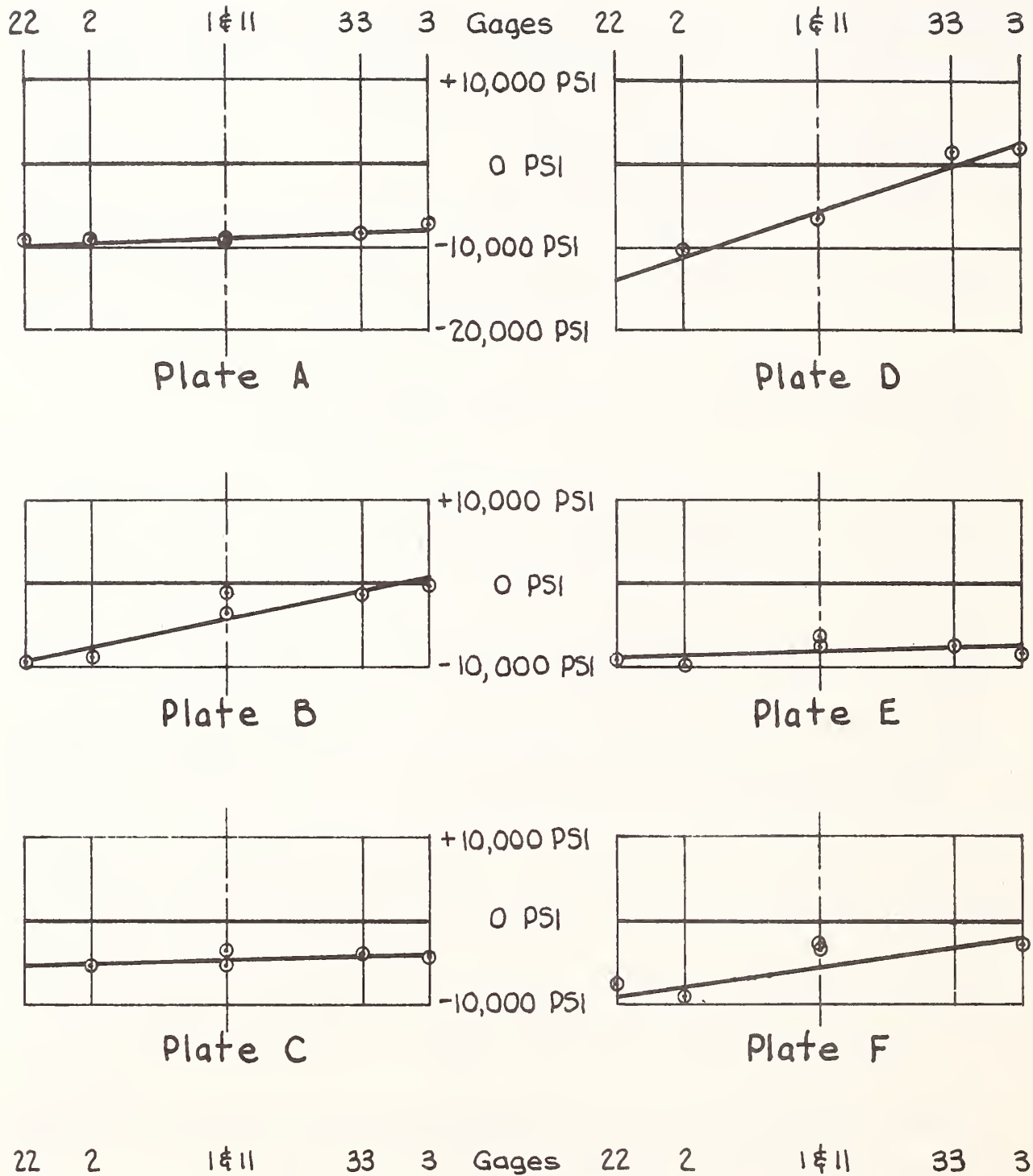


Figure 40. Stresses Caused by the Additional 78 Feet of Cover

TABLE 7. STRESSES CAUSED BY THE ADDITIONAL 78 FEET OF COVER-
1/4/66

Plate	Combined Stress psi	P/A Stress psi	MC/I Stress psi	Earth Pressure to Cause P/A psi	Bending Moment to cause MC/I** in lb/in
A	****	-8,700	****	38	****
B	9,800	-4,700	5,100	20	-1,000
C	5,800	-5,300	500	23	-100
D	12,000	-5,700	7,100	25	-1,400
E	9,600	-8,800	1,000	38	-200
F	9,300	-5,800	3,500	25	-700

** See footnotes on pages 68, 72 and 73

The results presented are only samples of the large quantity determined from the SR-4 strain gage installations. The output from the strain gage computer programs provided stress, earth pressure, and bending moment data for each of the approximately 52 dates on which a set of strain gage readings were taken. Samples of the computer output are given in Appendix A and the complete output, along with all original data, is on file in the Department of Civil Engineering and Engineering Mechanics at Montana State University. The data of January 4, 1966, was selected for presentation because the loads on the culvert were near their peak values and most of the SR-4 gages were still functioning properly at that time.

The results from some of the strain gages , after January of 1966 , began to deviate significantly from the linear pattern established earlier . At first it was theorized that this might be due to localized bending , longitudinal stresses , or general strain gage deterioration . It eventually became apparent however , that the first lot of gages , with a gage factor of 2.13 , were continuing to behave in a highly consistent and linear fashion while the gages from the second lot , with a gage factor of 2.08 , steadily became more erratic and inconsistent . The superior behavior of the first lot of gages was recognized because plates A and B continued to exhibit highly consistent strain patterns , much better than the other four plates . Plates A and B were unique in that all of the gages on Plate A , and all but one of the gages on Plate B , came from the first lot of gages procured for the project . Each of the other four plates had three gages from the first lot on the inside wall (gages 1 , 2 , and 3) and three gages from the second lot on the outside wall (gages 11 , 22 , and 33) . Examination of the strain data revealed that , in general , the gages from the first lot were giving consistent readings while those from the second lot were far more variable . Six months after installation most of the second lot of gages were completely unreliable . Most of the first lot still gave consistent strain readings after more than 18 months of operation . This may be seen from Figure 41 which shows stress patterns

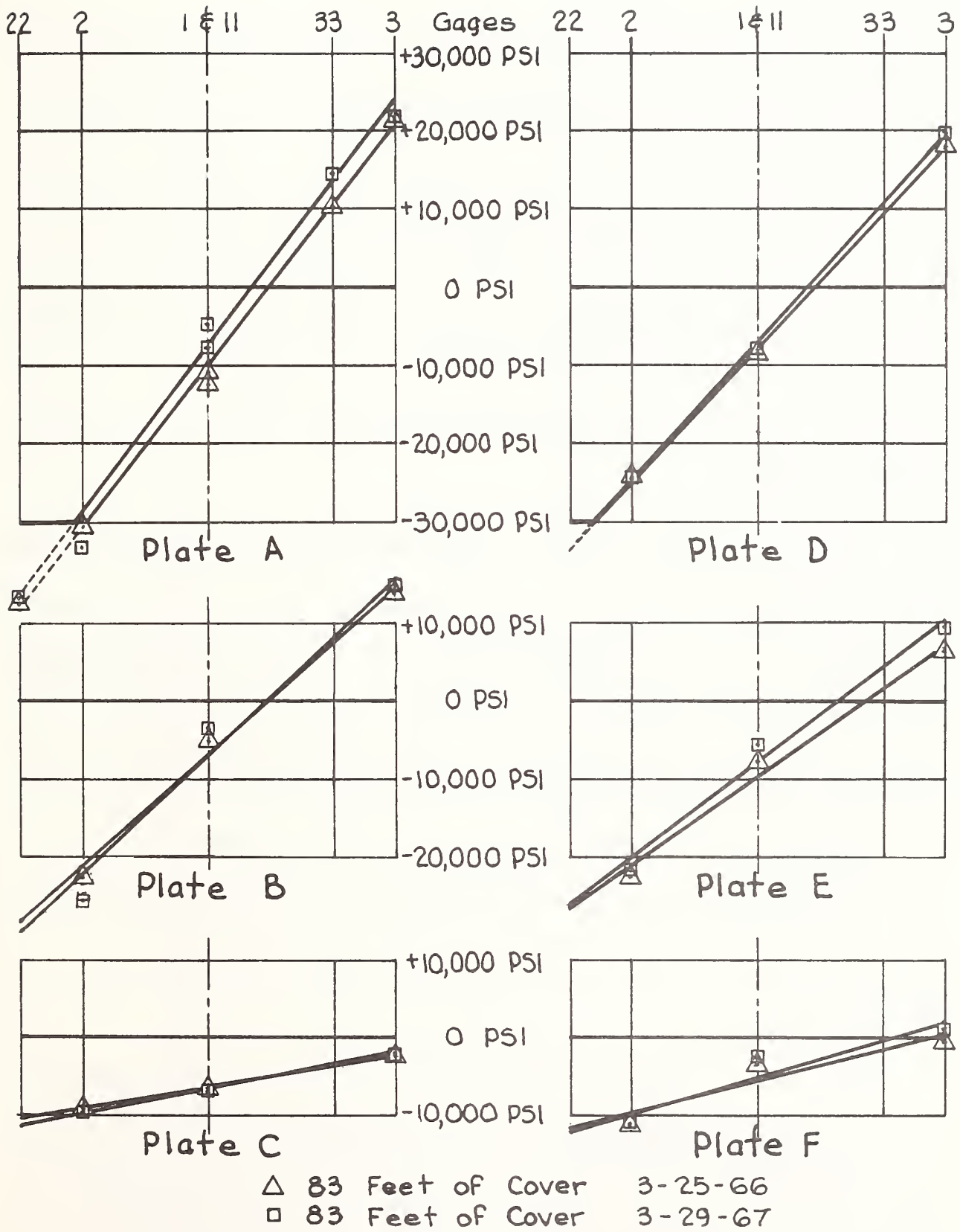


Figure 41. Combined Stresses on 3/25/66 and 3/29/67

deduced from the first lot of gages on widely separated dates.

There has been no satisfactory explanation for the consistently superior behavior of the first lot of gages. It is true that the second lot of gages are all on the outside walls in contact with the fill (but protected by angle iron houses) which may be related to their poor performance. It is considered highly significant, however, that gages from the first lot, in similar external positions on plates A and B, did not deteriorate similarly.

After the computer program was developed to reduce the strain gage data, a complete analysis was made using only the strain data from first lot gages, 1, 2, and 3, on each plate. The results for the earlier dates were essentially the same as when all of the gages were used. For the dates following completion of the embankment it was apparent that the second lot of gages should be excluded from the analysis. A decision was made to utilize the readings obtained from the 1, 2, and 3 numbered gages as the primary strain gage readings for calculating earth pressure and bending moments.

The equivalent vertical earth pressures on the culvert, calculated from the strain gage readings, are shown graphically in Figure 42 which indicates, in general, that the earth pressure reached a maximum in January or early February, 1966. The pressures decreased during the

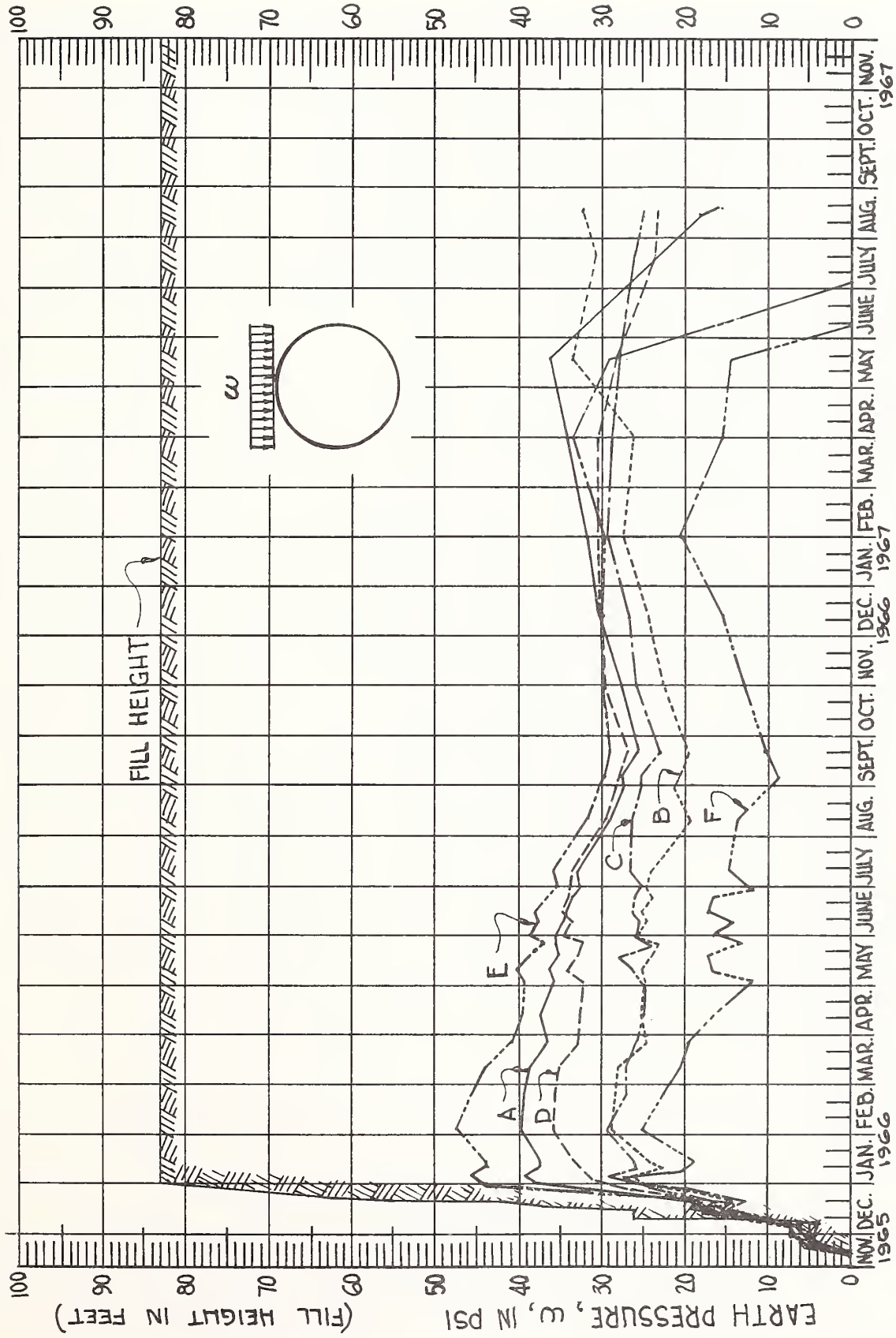


Figure 42. Equivalent Uniform Vertical Earth Pressures on Culvert, Deduced From SR-4 Strain Gages

winter and spring and eventually stabilized at a value near 30 psi. There is an indication of a slight upward trend in pressure starting in September of 1966. The maximum earth pressures ranged from 28 to 47 psi which is much less than the estimated nominal overburden pressure of 72 psi, or 10 ksf (kips per square foot), which one might expect from 83 feet of cover weighing approximately 125 pcf.

Plate F should be considered separately from the other 5 instrumented plates. Figure 42 shows that plate F indicated a substantially smaller earth pressure than the remaining 5 plates and this is entirely consistent with expectations because plate F is under the sloping side of the embankment at a place where the vertical depth of cover is only 45 feet (as scaled from Figure 22) as compared to the 83 feet depth over the central part of the culvert where plates A, B, and E are located. Plates C and D may be grouped with A, B, and E because they are under almost the full depth of cover near the intersection of the top of the embankment and the side slope.

General deterioration of the strain gages occurred in the summer of 1967 and, as Figure 42 shows, only three of the plates were still giving plausible results when reading were terminated in August.

Stress Meters

The earth pressures on the culvert walls as determined directly from the Carlson soil stress meters as the fill was constructed, and thereafter, are shown graphically as earth pressure versus time in Figures 43 through 48. The stress meters are grouped by their relative locations as measured

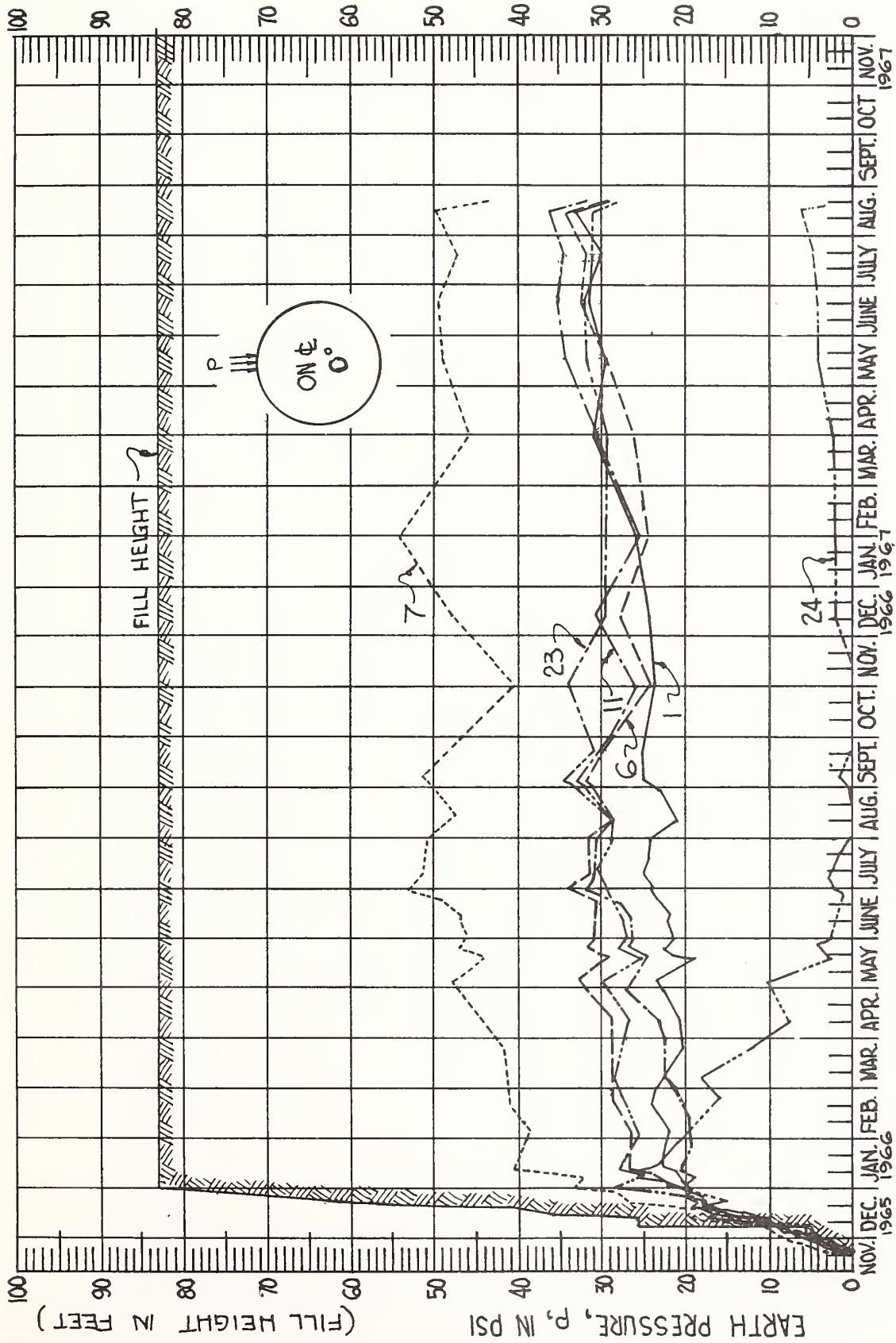


Figure 43. Earth Pressure vs. Time - Stress Meters No. 1, 6, 7, 11, 23 and 24

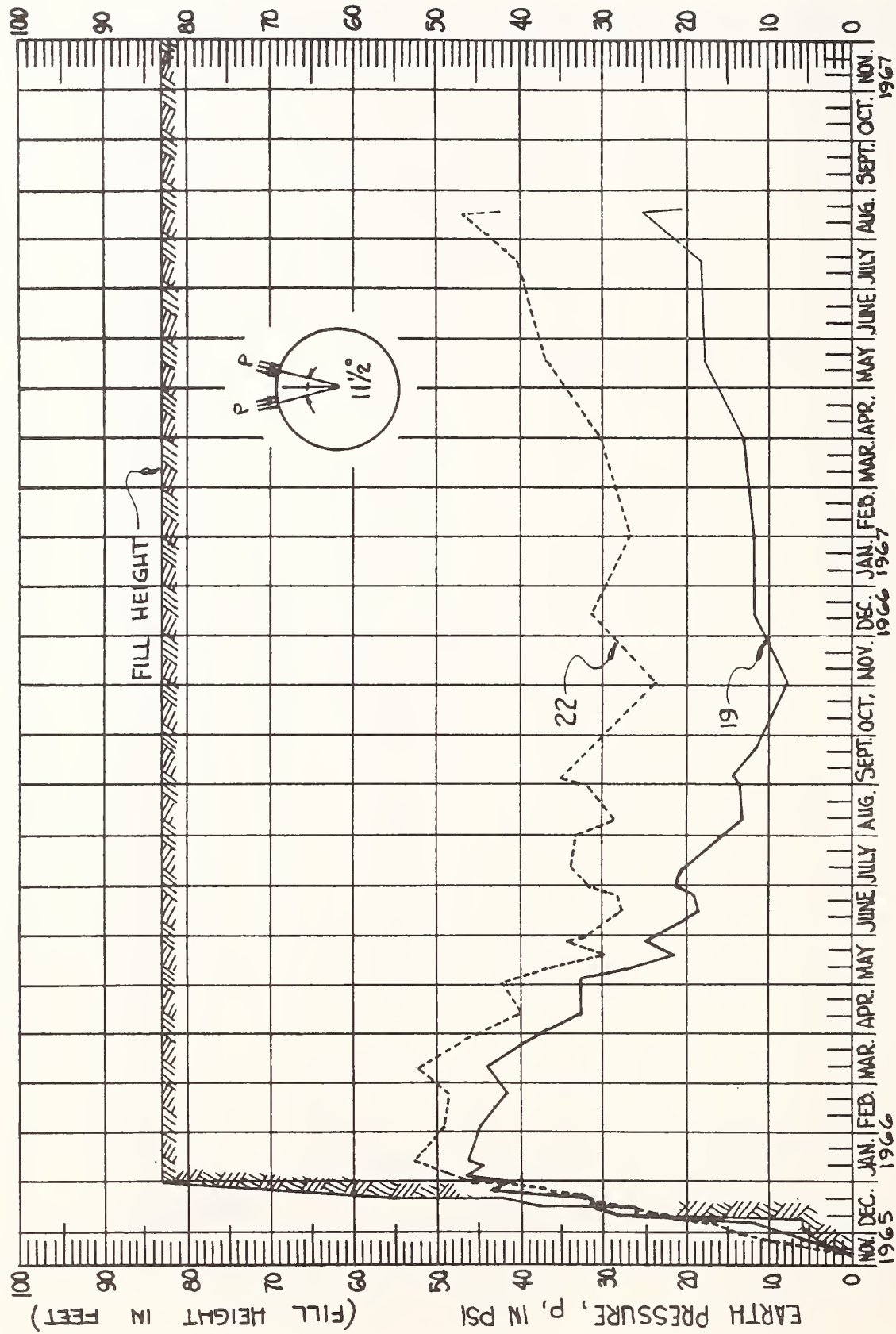


Figure 44. Earth Pressure VS. Time - Stress Meters No. 19 and 22

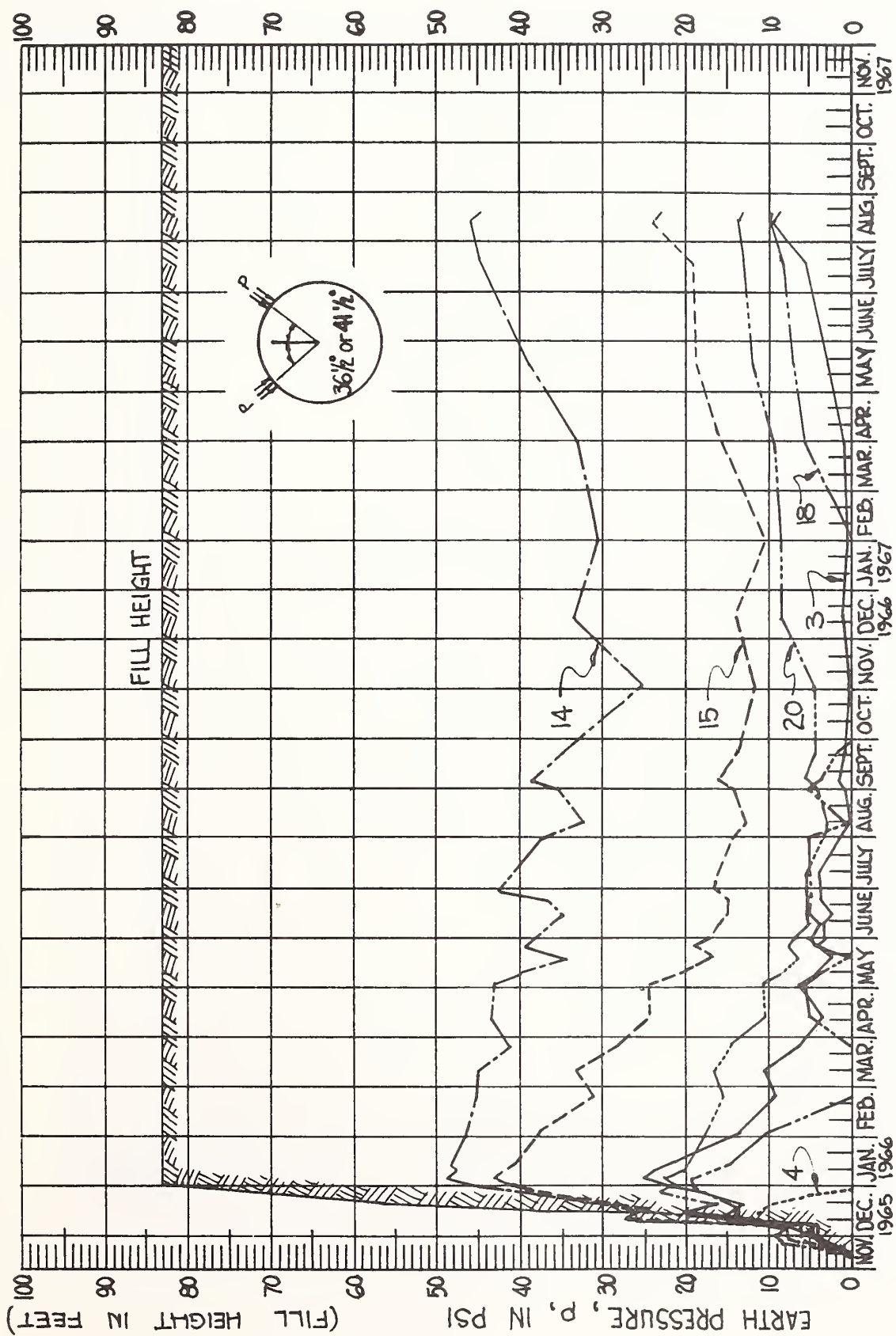


Figure 45. Earth Pressure VS. Time - Stress Meters No. 3, 4, 14, 15, 18 and 20.

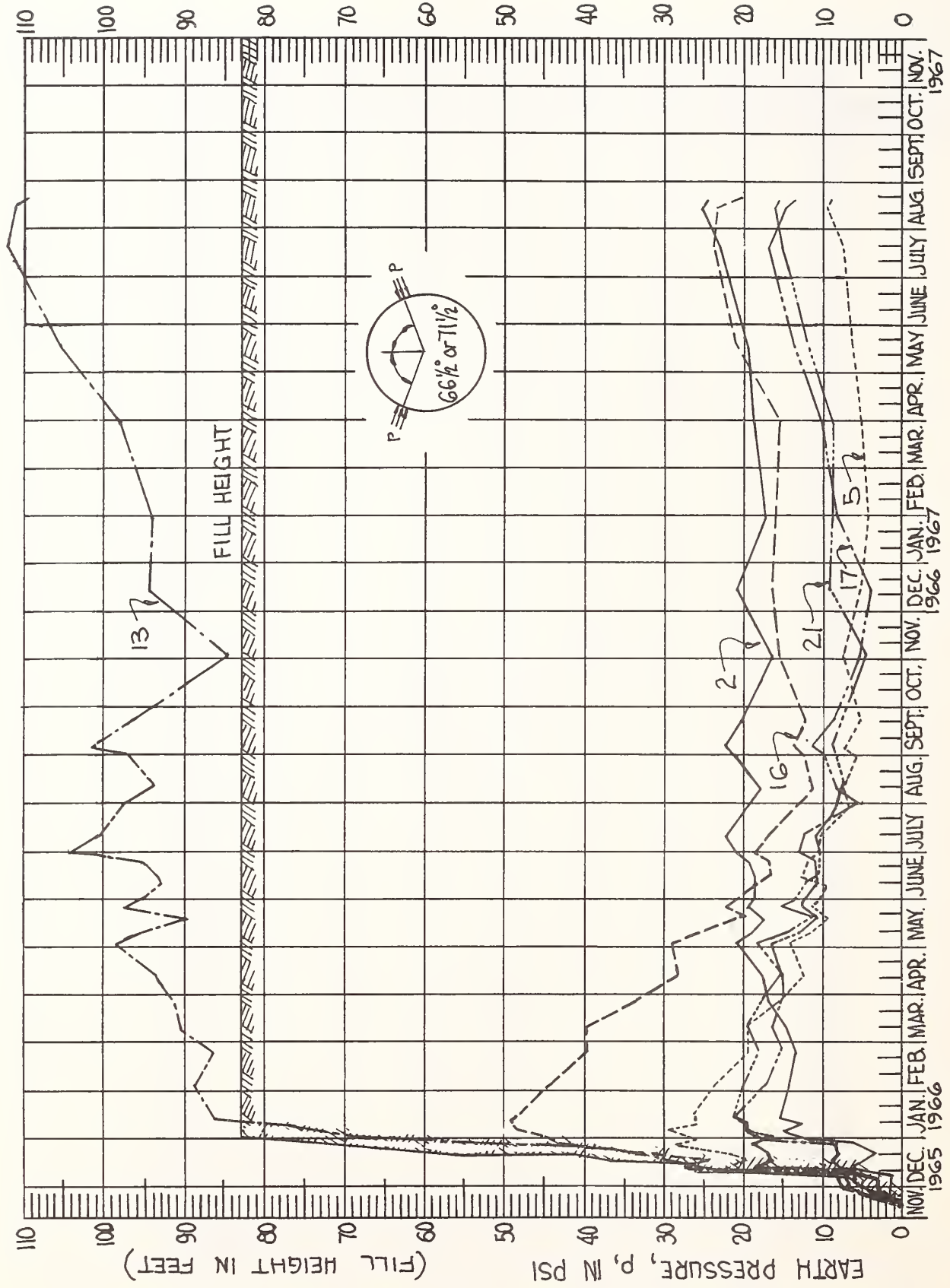


Figure 46. Earth Pressure vs. Time - Stress Meters no. 2, 5, 13, 16, 17 and 21.

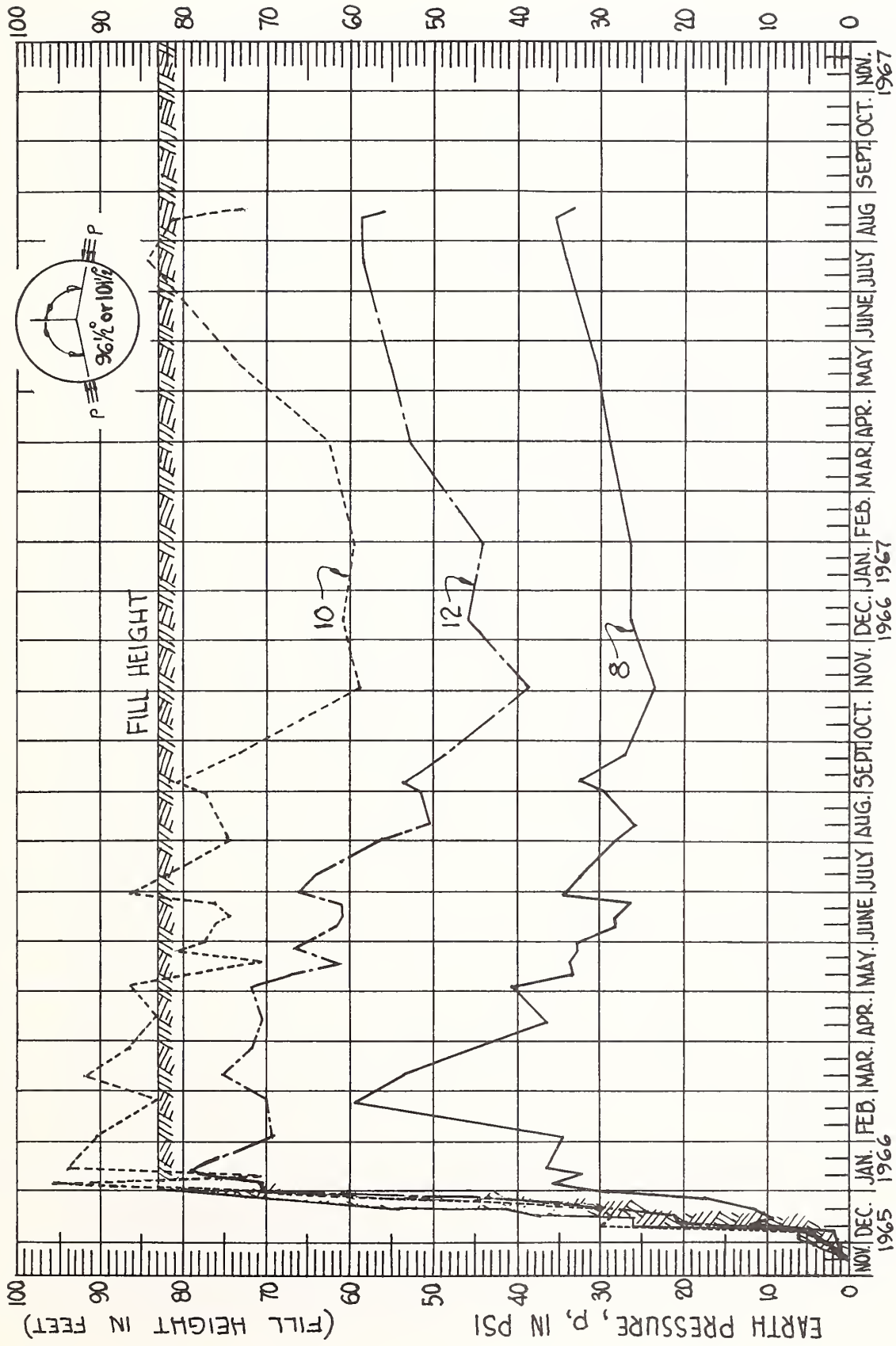


Figure 47. Earth Pressure vs. Time - Stress Meters no. 8, 10, and 12

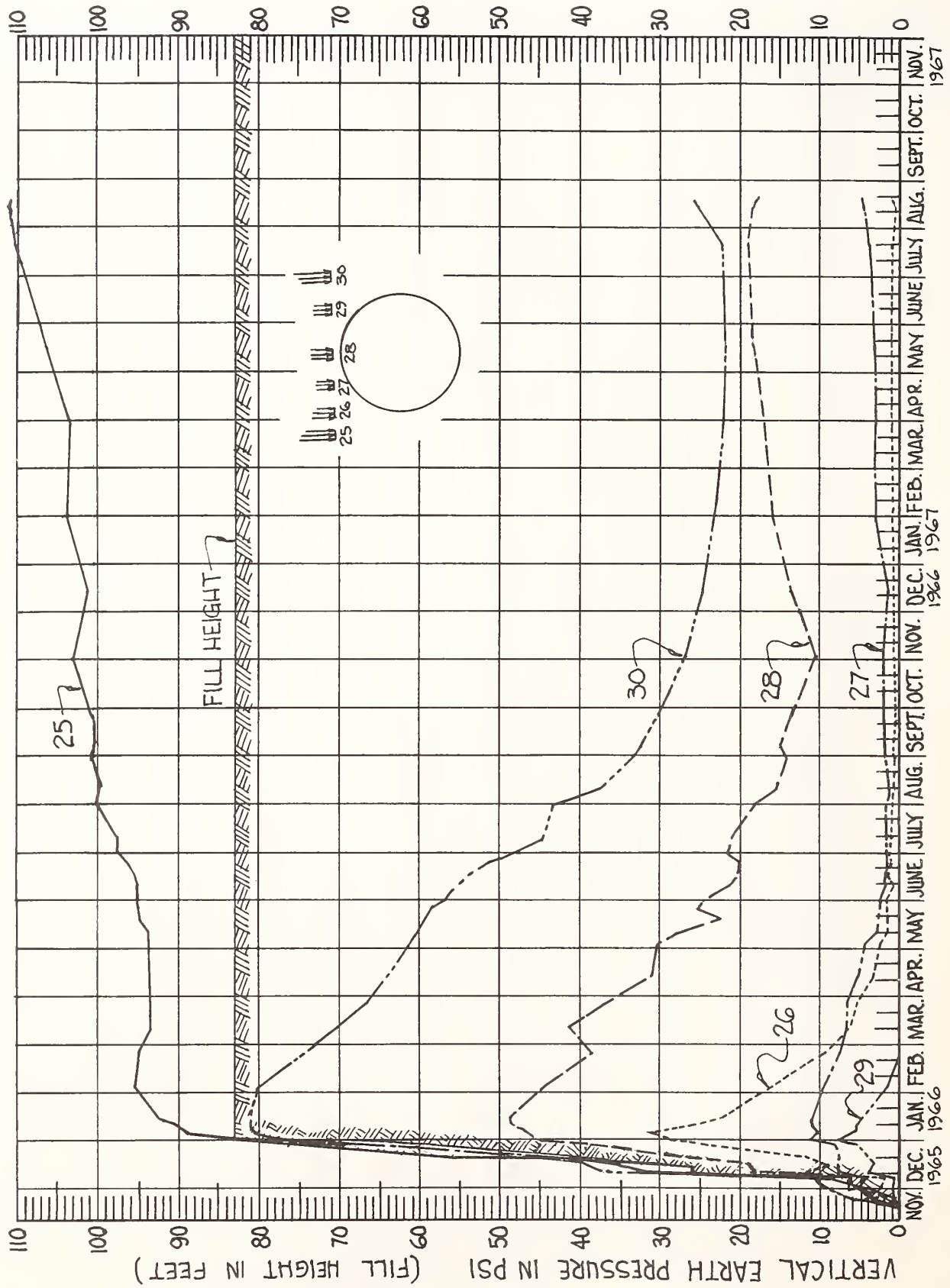


Figure 48. Earth Pressure vs. Time - Stress Meters no. 25, 26, 27, 28, 29 and 30.

in degrees from the top center line of the culvert. For example, meters 1, 6, 7, 11, 23, and 24 in Figure 43 are located on the top center line while meters 2, 5, 13, 16, 17, and 21 in Figure 46 are located on the culvert sides 66 1/2 or 71 1/2 degrees from the top center line. The fill height shown in these figures is the height of fill above the culvert, or the depth of cover.

The majority of the meters indicated a maximum pressure when the full fill height was reached or shortly thereafter. As a general trend, these pressures also decreased, as the earth pressures determined from the strain gages did, until the fall of 1966. A few of the meters did show an increase throughout the period covered by this report. It may be seen in Figure 43 that all of the meters located at the top of the culvert showed a small increase except for meter 24 which dropped to zero and started to indicate tension in August of 1966. It was concluded that meter 24 was defective, or had been damaged by construction equipment, and should be ignored when studying Figure 43. Figure 43 should also be studied in conjunction with Figure 29 which shows the approximate depth of cover above each meter. For instance, meter No. 1 is under the embankment slope and has a vertical depth of cover of approximately 45 feet; while meter 6 has approximately 63 feet of vertical cover. The effect of the varying depth of cover is not particularly apparent in Figure 43 which

shows remarkably close agreement among meters 1, 6, 11, and 23. The somewhat higher pressure registered by meter 7 is probably indicative of the rather large local pressure variation that could be expected from point to point on the top of a large culvert. Considering the small size of the pressure measuring area of a stress meter, the tight grouping of meters 1, 6, 11 and 23 is rather more surprising than the moderate deviation of meter 7.

The agreement between Figures 42 and 43, obtained with totally independent sets of distinctly different instrumentation, is gratifying. It should be kept in mind, however, that Figure 42 gives an average vertical pressure over the width of the culvert while Figure 43 gives only the vertical pressure at the top centerline of the culvert.

Figures 44, 45, 46, and 47 give the earth pressure on the sides of the culvert as measured with the stress meters. The general trend is the same as that of Figure 42, a decrease in pressure through the spring and summer of 1966, followed by a leveling off or a slight increase in pressure starting in the autumn of 1966.

When the pressures were largest, shortly after construction of the embankment, the pressures on the sides of the culvert, in some places at least, were considerably larger than the pressures on top. This is evident in Figure 49 which shows the stress pattern furnished by the

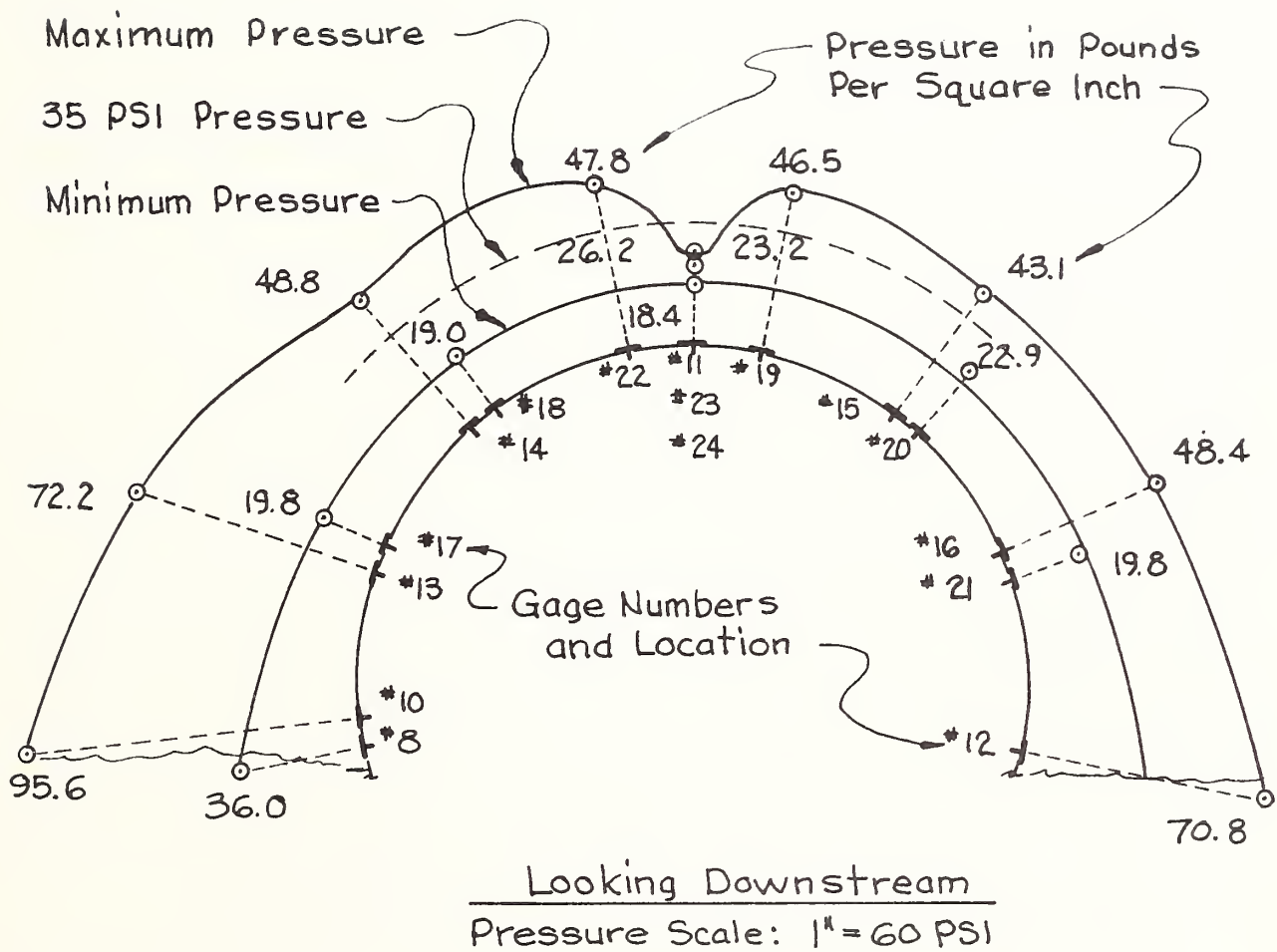


Figure 49. Earth Pressures Acting on the Culvert Walls on 1/4/66

stress meters on the culvert walls under the central part of the embankment on January 4, 1966.

From Figures 43 through 48 it is apparent that several of the meters, notably 3, 4, 18 and 29, were operating defectively in the same fashion as meter 24. Meters on the sides of the culvert were especially susceptible to sideswiping by construction equipment in the early stages of backfilling. Actually, a damaged meter that is leaking would be expected to show a steadily decreasing pressure. Such a meter might also be expected to register an abnormally low pressure right from the start. Meter 17, for example, is known to have been scraped by a truck tire (see Figure 26); and the low pressures reported by that meter, and shown in Figure 46, could be erroneous. It is suspected that many, if not all, of the meters registering very low pressures are damaged. It is concluded, however, that meters such as 8, 10, and 12 (see Figure 47) are not leaking because they show a pressure increase starting in the autumn of 1966.

Lateral pressure against the stem of a stress meter could cause highly erroneous readings. In a few cases there probably is contact pressure between the stem of a meter and the edge of the hole in the wall through which the stem protrudes into the culvert. Figure 27 shows how the stem of each meter protrudes into the culvert. The holes, which are somewhat irregular

because they were cut with a cutting torch, should have been made somewhat larger to entirely eliminate the danger of accidental contact between the stem and the side of the hole. Another possible source of lateral force on a stress meter stem is from tension in the electric cable connected to the stem. All of the cables were strung with a large amount of slack and checked periodically to see that they were not applying significant forces on the stems of the stress meters.

Overall, the performance of the stress meters is considered satisfactory. A conservative interpreter of the results might ignore all readings of less than about 20 psi. Figure 49 effectively summarized the pressure distribution on the culvert, as indicated by the stress meter readings of January 4, 1966. The difference between the maximum and minimum readings could be interpreted as random variation in pressure from point to point, or as experimental error, or as a combination of both. In any case, Figure 49 indicates that the average pressure over the roof area of the culvert was probably close to 35 psi on January 4, 1966, but somewhat more than 35 psi on the sides. A re-examination of Figure 42 shows that the SR-4 gages also indicated an average vertical pressure close to 35 psi on that date. (The actual numerical average of the pressures indicated by Plates A, B, C, D and E on that date is 34 psi).

Primarily as a result of the demonstrated good agreement between

the two major sets of instrumentation, it is concluded that both sets gave reasonably reliable or accurate results.

Figure 48 gives the pressures from the six stress meters that were buried in the fill to measure vertical earth pressure in the earth at an elevation one foot above the top of the culvert. Construction equipment could have overstressed some of these meters before the depth of fill above them could protect them from high local wheel loads. An attempt was made to keep all vehicles off the immediate area until the meters were covered to a depth of one foot but it appears that meters 26, 27, and 29 were damaged, and perhaps meters 28 and 30 as well. The readings, on January 4, 1966, are plotted on Figure 50. Taken collectively these meters indicate an average vertical pressure above the culvert which is in the vicinity of 30 psi, which constitutes good general agreement with the meters on the culvert walls and the SR-4 strain gages, and suggests that the damaged meters were still giving fairly accurate readings at that early date.

The relatively very high earth pressures on meters 25 and 30, well above the estimated nominal overburden of 72 psi, constitutes strong independent evidence that the "imperfect trench" construction detail was working in the manner visualized by those who conceived it and actively promulgated it as a practical way to relieve the vertical pressure on culverts. If the "imperfect trench" relieves the pressure on the culvert it

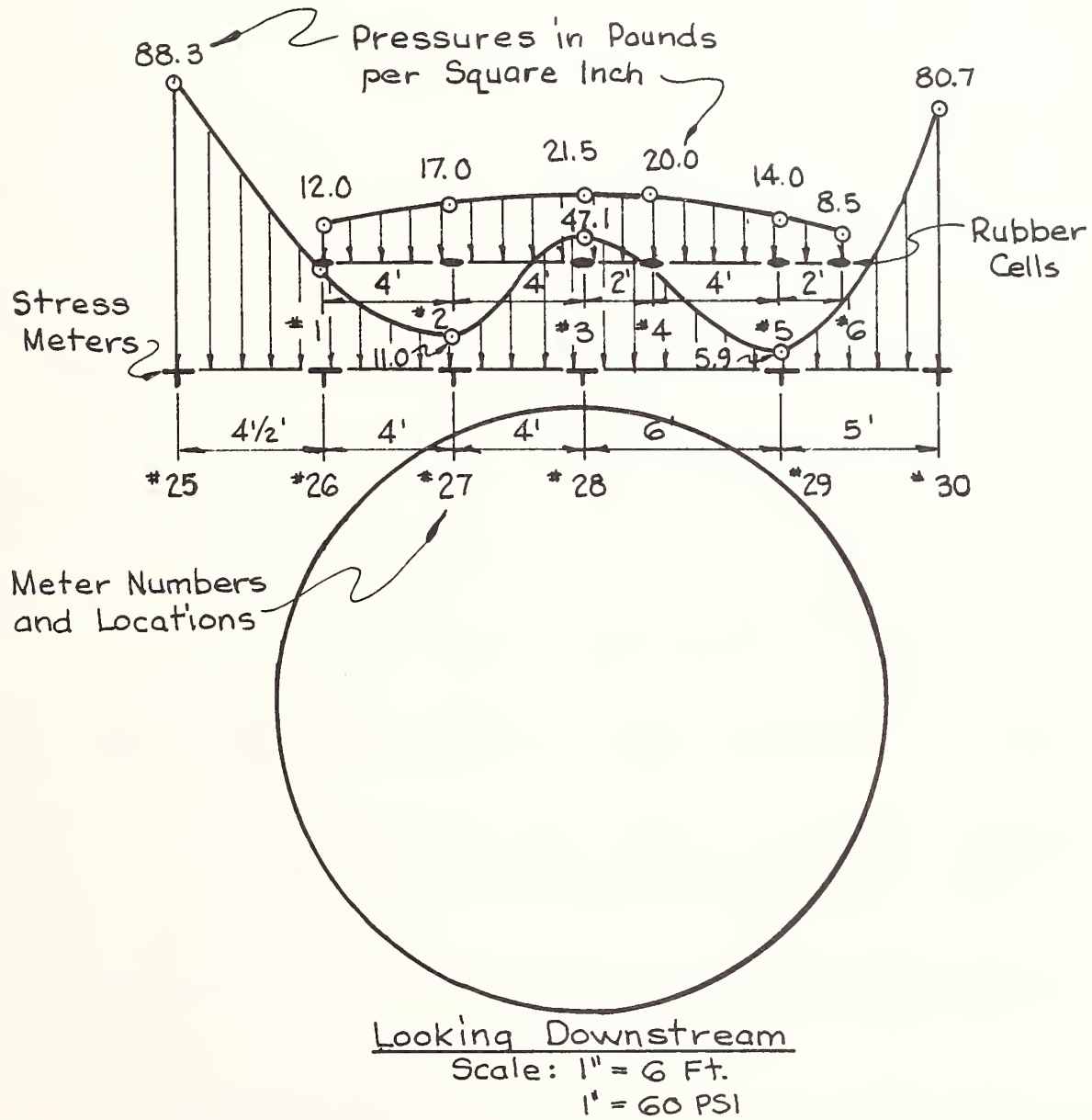


Figure 50. Earth Pressures on 1/4/66 Determined from the Stress Meters and Rubber Pressure Cells Located in the Fill

must, necessarily, increase the pressure on the earth adjacent to the culvert because the weight of the overburden must be supported somewhere.

Rubber Pressure Cells

Still another set of independent pressure-measuring instrumentation was the set of six rubber diaphragm pressure cells that were installed six inches below the straw, above ring 38 of the rebuilt culvert. As expected, these cells registered substantially lower pressures than the other instrumentation because they were placed under the straw. The knowledge that they might have burst under heavy earth loads or construction equipment loads was the primary motivation for placing them in the highly cushioned location that was chosen for them.

Cell No. 2, which was connected to the center Bourdon gage shown in Figure 31, developed a visible leak at the site of the bleeder valve below the Bourdon gage and the pressure on this cell inevitably dropped off steadily as fluid leaked out. The other five cells appeared to function flawlessly except for a very minor leak that developed between the shutoff valve and the Bourdon gage of cell No. 6. For this particular cell, the shutoff valve was kept closed between pressure readings, and was opened for no longer than half a minute at a time to take a reading on the Bourdon gage. No more than a small fraction of a cubic centimeter of liquid es-

caped from cell No. 6 and it has seemingly functioned accurately from the beginning.

Figure 51 provides a graphical record of the pressures registered by the rubber pressure cells. The pressure variations should not be interpreted as random scatter because all of the pressure variations, from cell to cell, is demonstrably a function of cell location. This becomes apparent when the pressures for any given day are plotted on a cross-section at the location of each cell. Figure 50 shows these pressures, as of January 4, 1966, in addition to pressures from the buried stress meters. For the sake of efficiency, both pressure diagrams are shown plotted on the same cross-section diagram but the reader should bear in mind that the rubber pressure cells are approximately 30 feet downstream from the buried stress meters as well as approximately 3 1/2 feet above them.

In Figure 50, the plotted pressures for the rubber cells define a smooth and nearly symmetrical pressure diagram. The maximum pressure is at cell No. 3, right at the center, with a smooth transition to a minimum pressure on each side, at cells 1 and 6 which are located under the outermost bales of straw at the locations of anticipated minimum pressure. According to the rubber pressure cell readings, the average vertical pressure at the base of the straw was approximately 17 psi on January 4, 1966.

The loose rock fill above the straw arches between the "walls" of fill

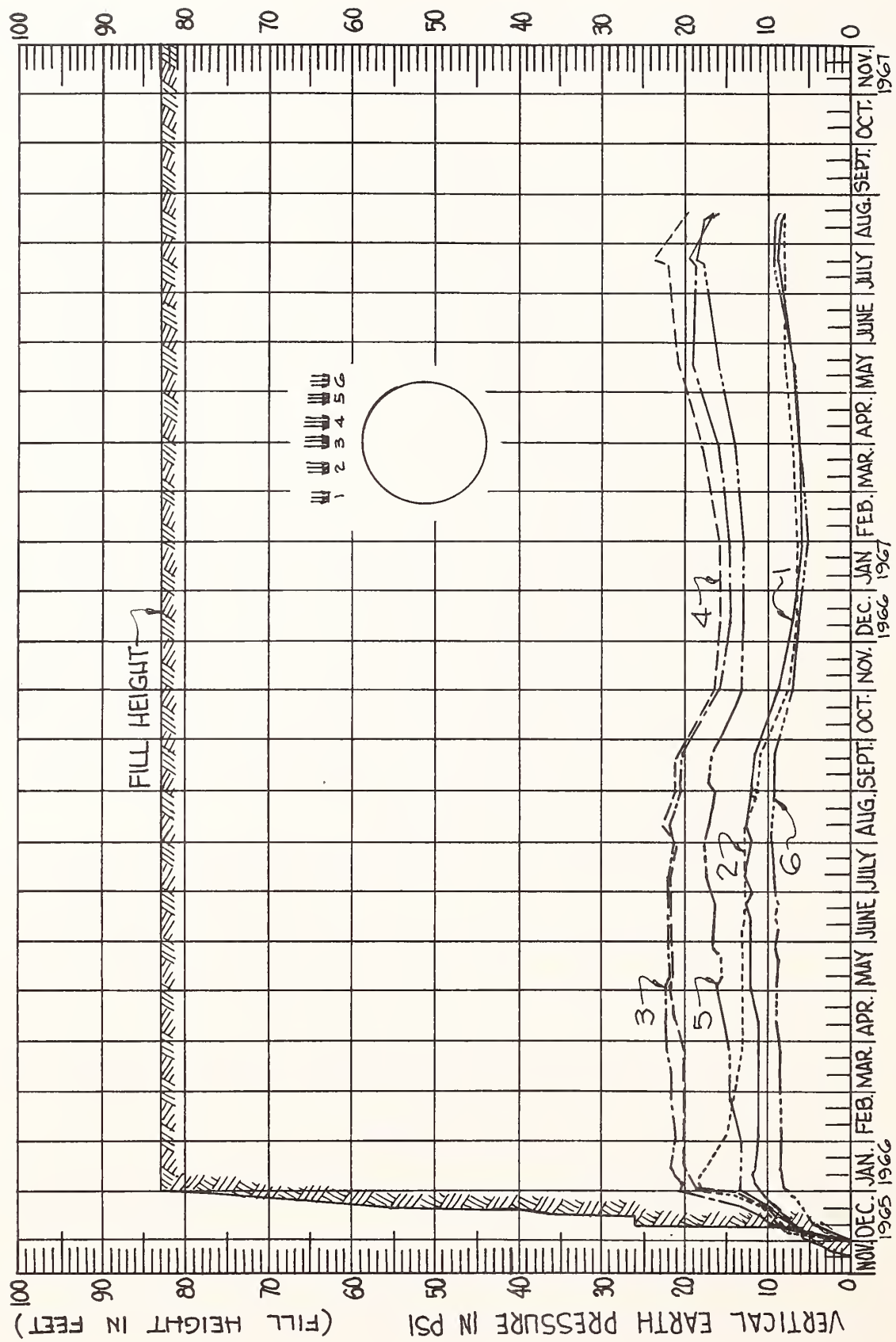


Figure 51. Earth Pressures vs. Time - Rubber Pressure cells no. 1, 2, 3, 4, 5 and 6

on each side of the straw (refer back to Figure 2) and the weight of the arching fill is supported largely by this arching action, except for the lowermost portion of the loose fill near midspan which is too close to the straw to be supported by the arch above it and therefore bears directly against the straw in the midspan region and causes the pressure on the straw to be greater at midspan than elsewhere.

Settlement Cells

Representative observed settlements of the eight settlement cells located in the fill on each side of the culvert, and slightly above the elevation of the top of the culvert, are given in Table 8. The designated central segment of the table encompasses the period during which apparent frost problems and varying densities of the liquid in the opposing legs of the manometer systems produced some obviously inconsistent and inaccurate settlement readings. In spite of several glaring inaccuracies, the central part of the table still indicates an average trend which is consistent with the earlier and the later settlements. The bottom part of the table, which shows a large number of settlement readings taken during a time period when settlement was occurring very slowly, and after the source of the earlier erratic readings was identified and eliminated, provides a good indication of the reproducibility or inherent potential accuracy of the simple U tube manometer system of the

TABLE 8. SETTLEMENT OF THE SETTLEMENT CELLS IN INCHES

Date	Cell Number and Location (North, South)							
	1 S	2 N	3 S	4 S	5 N	6 S	7 S	8 N
	inches							
12/ 5/65	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12/ 7/65	2.10	0.50	0.80	0.70	0.40	0.70	0.60	0.45
12/ 8/65	2.10	1.00	1.35	1.30	0.80	1.15	1.05	0.85
12/13/65	2.80	1.40	2.10	2.10	1.30	1.85	1.70	1.50
12/15/65	2.80	1.50	2.20	2.35	1.55	2.20	1.80	1.90

12/22/65	3.20	2.00	2.70	3.30	2.15	3.50	2.60	2.70
12/30/65	2.80	1.40	3.45	3.70	1.70	3.20	2.80	2.50
1 /13/66	2.70	1.10	3.55	4.05	1.65	3.25	2.80	2.45
2 / 3/66	2.90	1.60	4.00	4.70	2.40	3.85	3.30	3.00
3 /10/66	3.90	1.55	4.30	5.20	2.20	3.80	3.20	2.90
5 /31/66	4.45	2.95	5.10	5.40	2.95	4.55	4.80	3.65

6 /14/66	3.40	2.00	4.45	4.70	2.35	3.75	3.90	3.00
6 /23/66	3.45	2.05	4.40	4.85	2.35	3.80	3.95	3.05
8 / 9/66	3.40	1.95	4.45	4.80	2.25	3.75	3.95	3.00
9 /21/66	3.35	1.90	4.50	4.80	2.25	3.77	4.00	3.00
12/13/66	3.60	2.05	4.65	4.95	2.40	4.00	4.15	3.15
3 /29/67	3.70	2.10	4.70	5.05	2.45	---*	4.20	---*
5 /17/67	3.45	1.90	4.60	4.90	2.25	3.90	4.05	3.05
7 /18/67	3.75	2.15	4.75	5.20	2.40	4.10	4.30	3.05
7 /20/67	3.70	1.90	4.55	4.85	2.30	3.95	4.15	2.95
8 /14/67	3.55	1.85	4.50	4.85	2.20	3.95	4.10	3.05
8 /16/67	3.55	1.85	4.50	4.85	2.15	3.90	4.15	3.10
8 /17/67	3.70	2.10	4.70	5.05	2.40	4.10	4.25	3.20

--- --- --- Readings within this section are unreliable.

*The lines were plugged.

settlement cells.

Settlement cells number 2, 5, and 8 were located on the north side of the culvert and cells 1, 3, 4, 6, and 7 were on the south side. The readings were initially taken by adding fluid, as previously mentioned, and then letting the system stabilize, but was later changed to draining the system and refilling it with the pump each time a settlement reading was desired. This method was begun in June of 1966. Some of the readings were obviously very much in error, notably all of the readings taken on May 31, 1966. It was early in June of 1966 before it was discovered that the rather carelessly proportioned half-and-half mixture of water and Prestone that was periodically poured into the standpipes on the manometer board was causing a difference in the specific gravities of the liquid in opposing legs of the manometers. The researchers were priming the standpipes with a too-rich mixture of Prestone and water which was progressively depressing the equilibrium level in the standpipes because Prestone is significantly more dense than water. All of the settlement readings taken on and after June 14, 1966, indicate that the preceding readings showed fictitiously large settlements.

The average settlement of the settlement cells is illustrated in Figure 52 and plotted as settlement versus time. Settlement cells No. 5 and 8 on the north side and settlement cells No. 3, 4, 6, and 7, on the south side are used for this purpose. The cells were installed when there was five feet of

cover and there was no indication of settlement until after fill was placed on the straw. The reference line shown refers to the top of the culvert, in the vicinity of the settlement cells, immediately after their installation.

In Figure 52, the settlement of the settlement cells should be compared only to that portion of the settlement of the top of the culvert which occurred after December 6, 1965. For example, the settlement of the top of the culvert, between December 6, 1965, and August, 1967, as measured from point B in Figure 52, is approximately 2.2 inches. During the same time interval, the settlement of the north side settlement cells averaged 2.7 inches (as measured from point A) while the settlement of the south side settlement cells averaged 4.5 inches. These figures show that the soil on each side of the culvert settled more than the top of the culvert or the soil directly above the culvert. In doing so, the soil on each side would, necessarily, subject the intervening soil (directly above the culvert) to downward frictional drag forces.

Culvert Invert Settlement

The settlement of the culvert invert was determined periodically with an engineer's level and partial results are given in Table 9. This table shows the elevation of the iron pins in the control devices immediately after erection, and the total settlement of these pins in feet at various dates thereafter as calculated from the level readings. The control device at station number one was damaged during the first winter and was not used after January of 1966.

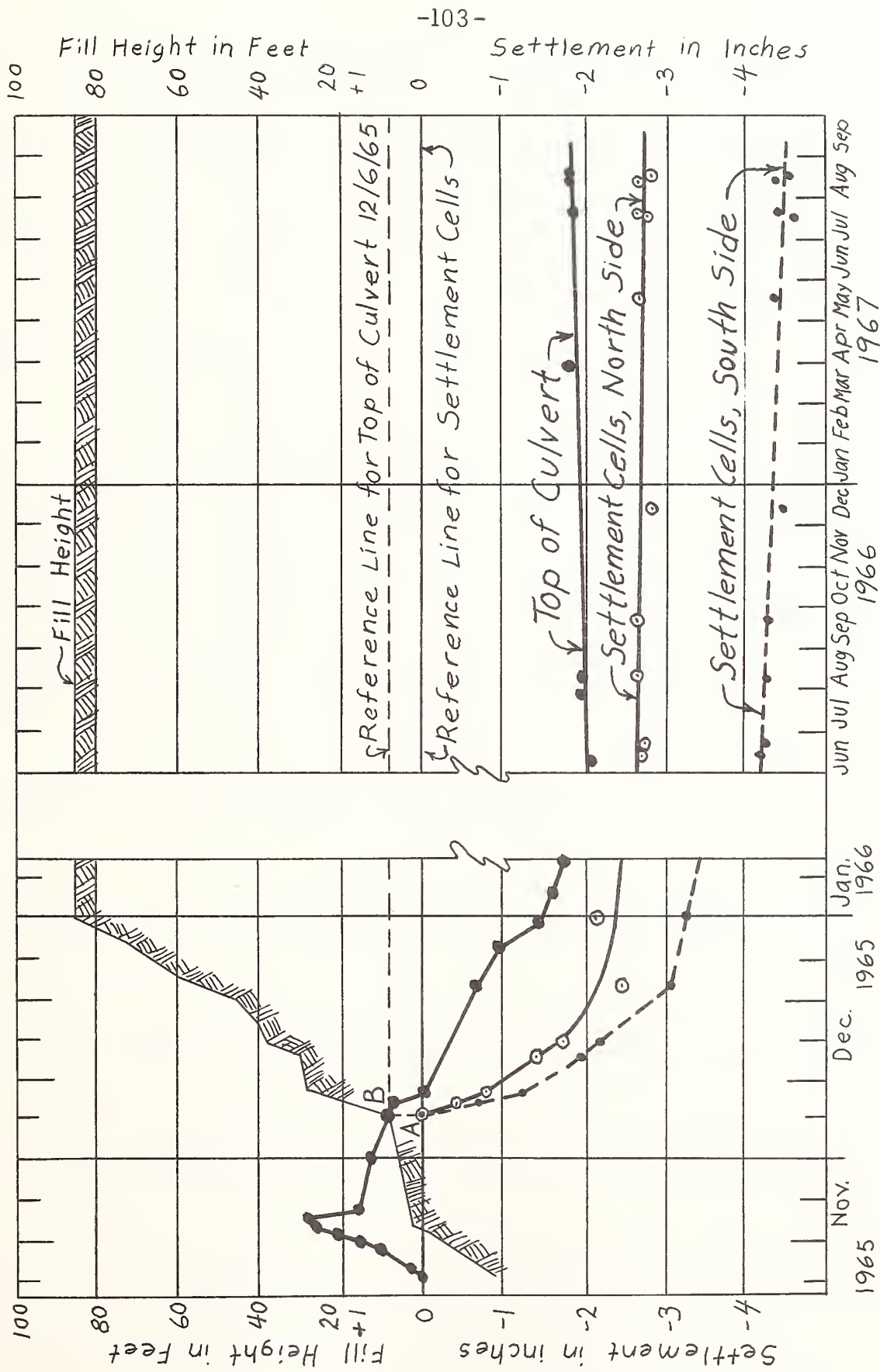


Figure 52. Settlement of the Settlement Cells and Top of Culvert

TABLE 9. SETTLEMENT OF THE CULVERT INVERT

Station	Elevation	12/29/65 feet	1/13/66 feet	6/15/66 feet	6/23/66 feet	7/28/66 feet	8/15/67 feet
	after Erection						
1	3829.09	0.06	0.06	----	----	----	----
2	28.99	0.05	0.05	0.06	0.07	0.07	0.06
3	28.83	0.12	0.11	0.10	0.11	0.15	0.11
4	28.61	0.11	0.13	0.10	0.11	0.13	0.12
5	28.62	0.10	0.08	0.09	0.10	0.09	0.10
6	28.51	0.09	0.09	0.07	0.09	0.08	0.09
7	28.56	0.07	0.07	0.05	0.08	0.07	0.08
8	28.31	0.05	0.05	0.05	0.04	0.06	0.08
9	28.00	0.11	0.07	0.07	0.06	0.04	0.09
10	28.00	0.09	0.08	0.08	0.08	0.08	0.09
11	27.74	0.06	0.03	0.05	0.05	0.02	0.09
12	27.62	0.06	0.04	0.05	0.03	0.02	0.07
13	27.56	0.00	0.00	0.00	0.01	----	0.02
Average	-----	0.075	0.066	0.064	0.069	0.068	0.083

It is clear, from Table 9, that the settlement of the culvert invert was generally less than 0.10 feet and that practically all of the settlement had taken place by December 29, 1965. The variations in the table are not considered significant because they could all have been caused by variations in reading the level rod. Errors as large as 0.02 or 0.03 feet could easily have occurred as a result of poor light conditions inside the culvert and other difficulties associated with having the level set up in the streambed upstream from the culvert.

The total settlement of the culvert invert, in the middle portion of the culvert, may be taken as approximately 0.08 feet or 1.0 inches, without

any significant error. The downward settlement of the top of the culvert, as plotted in Figure 52 for all dates in 1966 and 1967, was obtained by adding 1.0 inches to the average vertical diameter change between stations 6 and 9, inclusive.

Diameter Changes

The change in the average vertical and horizontal diameter of the culvert, in inches, is illustrated in Figure 53, using stations 6 through 9 which were under the highest portion of the fill. The top portion of this figure shows the relationship between the top of the culvert and the fill height as the diameters changed. The fill height shown is in feet above the culvert. The backfilling began at an elevation approximately 19 feet below the top of the culvert. The bottom curves show the change in diameters. The figure shows that the vertical diameter increased by approximately 1.4 inches as the backfilling progressed until the culvert was covered, then decreased until it was approximately 1.0 inches less than the diameter just after erection, and then increased again, slightly. The horizontal diameter first decreased by approximately 2.1 inches and then began to increase after the culvert was covered. The horizontal diameter remains approximately 0.7 inches less than it was at the time of erection. Representative actual diameter changes at all 13 measuring stations are listed in Tables 10 and 11.

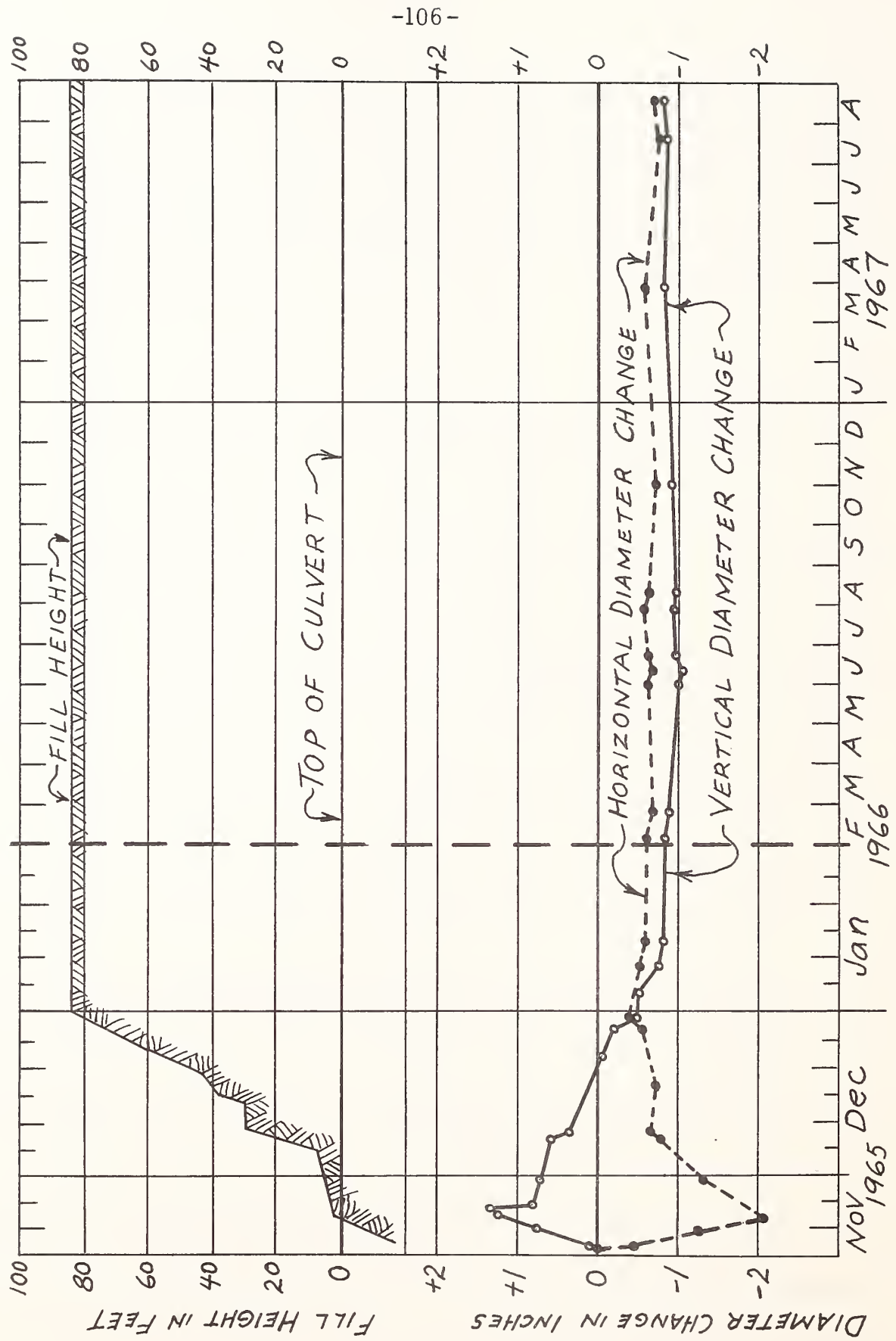


Figure 53. Average Horizontal and Vertical Diameter Change

TABLE 10. VERTICAL DIAMETER CHANGE IN INCHES

Date Sta.	Nov 16 1965	Nov 23 1965	Nov 29 1965	Dec 8 1965	Dec 29 1965	Jan 13 1966	Feb 3 1966	Jun 9 1966	Nov 1 1966	Mar 29 1967	Jul 20 1967	Aug 17 1967
1	0.00	0.88	1.13	0.50	0.36	0.20	0.40	-----	-----	-----	-----	-----
2	0.00	1.50	1.13	0.63	0.00	-0.20	-0.10	-0.45	-0.20	-0.05	-0.15	-0.10
3	0.00	1.50	1.25	1.13	0.66	0.40	0.40	0.40	0.60	0.90	0.70	0.75
4	0.00	1.63	1.25	1.13	0.48	0.10	0.05	0.00	0.30	0.35	0.30	0.40
5	0.00	1.25	0.88	0.50	-0.36	-0.60	-0.60	-0.65	-0.050	-0.30	-0.30	-0.20
6	0.00	1.25	1.00	0.50	-0.24	-0.65	-0.70	-0.80	-0.65	-0.50	-0.60	-0.50
7	0.00	1.50	0.88	0.50	-0.48	-0.95	-0.90	-0.95	-0.90	-0.85	-0.85	-0.75
8	0.00	1.63	0.50	0.25	-0.72	-0.95	-1.10	-1.15	-1.05	-1.05	-1.00	-1.05
9	0.00	1.13	0.50	0.25	-0.36	-0.65	-0.70	-1.20	-1.05	-0.85	-0.90	-0.85
10	0.00	1.88	1.00	0.63	-0.60	-0.45	-0.40	-0.50	-0.40	-0.20	-0.30	-0.25
11	0.00	1.33	0.75	0.50	-0.60	-0.55	-0.50	-0.65	-0.50	-0.25	-0.30	-0.25
12	0.00	1.50	1.00	0.63	-0.48	-0.55	-0.60	-0.60	-0.40	-0.30	-0.25	-0.20
13	0.00	1.50	0.88	0.38	-0.90	-1.00	-1.00	-0.70	-0.80	-0.70	-0.75	-0.70

TABLE 11. HORIZONTAL DIAMETER CHANGE IN INCHES

Date Sta.	Nov 16 1965	Nov 22 1965	Nov 29 1965	Dec 8 1965	Dec 29 1965	Jan 13 1966	Feb 3 1966	Jun 9 1966	Nov 1 1966	Mar 29 1967	Jul 20 1967	Aug 17 1967
1	0.00	-2.75	-1.50	-0.25	-0.12	0.00	-0.10	-----	-----	-----	-----	-----
2	0.00	-----	-0.63	-0.25	1.20	0.52	0.60	0.52	0.57	0.83	0.73	0.83
3	0.00	-2.00	-1.63	-1.00	-0.96	-1.10	-1.20	-1.15	-1.25	-1.15	-1.35	-1.25
4	0.00	-1.63	-1.50	-0.88	-0.60	-0.70	-0.75	-0.80	-0.85	-0.70	-0.90	-0.80
5	0.00	-2.00	-1.63	-1.63	-0.36	-0.60	-0.60	-0.65	-0.65	-0.55	-0.70	-0.65
6	0.00	-2.50	-1.63	-0.88	-0.60	-0.70	-0.70	-0.77	-0.88	-0.73	-0.93	-0.82
7	0.00	-1.88	-1.25	-0.75	-0.60	-0.73	-0.70	-0.78	-0.88	-0.73	-0.88	-0.87
8	0.00	-1.88	-1.25	-0.63	0.00	-0.70	-0.80	-0.82	-0.88	-0.68	-0.88	-0.77
9	0.00	-2.00	-1.00	-0.38	0.36	-0.15	-0.25	-0.25	-0.25	-0.15	-0.35	-0.25
10	0.00	-1.63	-1.13	-0.50	-0.12	-0.27	-0.10	-0.32	-0.37	-0.28	-0.43	-0.42
11	0.00	-1.63	-1.38	-0.50	0.00	0.03	-0.20	-0.18	-0.23	-0.13	-0.18	-0.17
12	0.00	-1.88	-1.25	-0.75	-0.12	-0.18	-0.30	-0.23	-0.38	-0.18	-0.38	-0.27
13	0.00	-2.25	-1.75	-1.13	0.00	0.05	-0.05	0.00	-0.05	-0.15	-0.05	0.05

$$\frac{1}{108}$$

Settlement of Roadway Pavement

Elevations were measured along the center-line of the paved roadway to determine if there was more settlement over the culvert than elsewhere. The initial levels were taken in September of 1966 after the surfacing was completed. As stated previously, the fill was completed in January, 1966, and the surfacing was begun in August, 1966, and during this time the roadway was open to traffic. Since the levels were not taken until the surfacing was completed, a large portion of the embankment settlement undoubtedly had taken place by then.

Figure 54 shows the settlement of the roadway between September, 1966, and August, 1967. This figure shows that the settlement of the pavement over the culvert was not more than the settlement of the pavement on each side of the culvert. In fact, there was more settlement of the pavement 100 feet south of the culvert than directly above the culvert. The largest settlement was 0.14 feet at station 129 plus 55.

The paving was continuous across all four lanes and the narrow median strip, and the elevation measurement stations were situated on the highway centerline between the median guard-rail posts. An interesting sidelight of the settlement measurements is that they revealed that a slight amount of upward bulging of the highway centerline occurred between March and August of 1967. This is apparent in Figure 54. The upward movement appears to be associated with a slight amount of rutting in the wheel paths on each

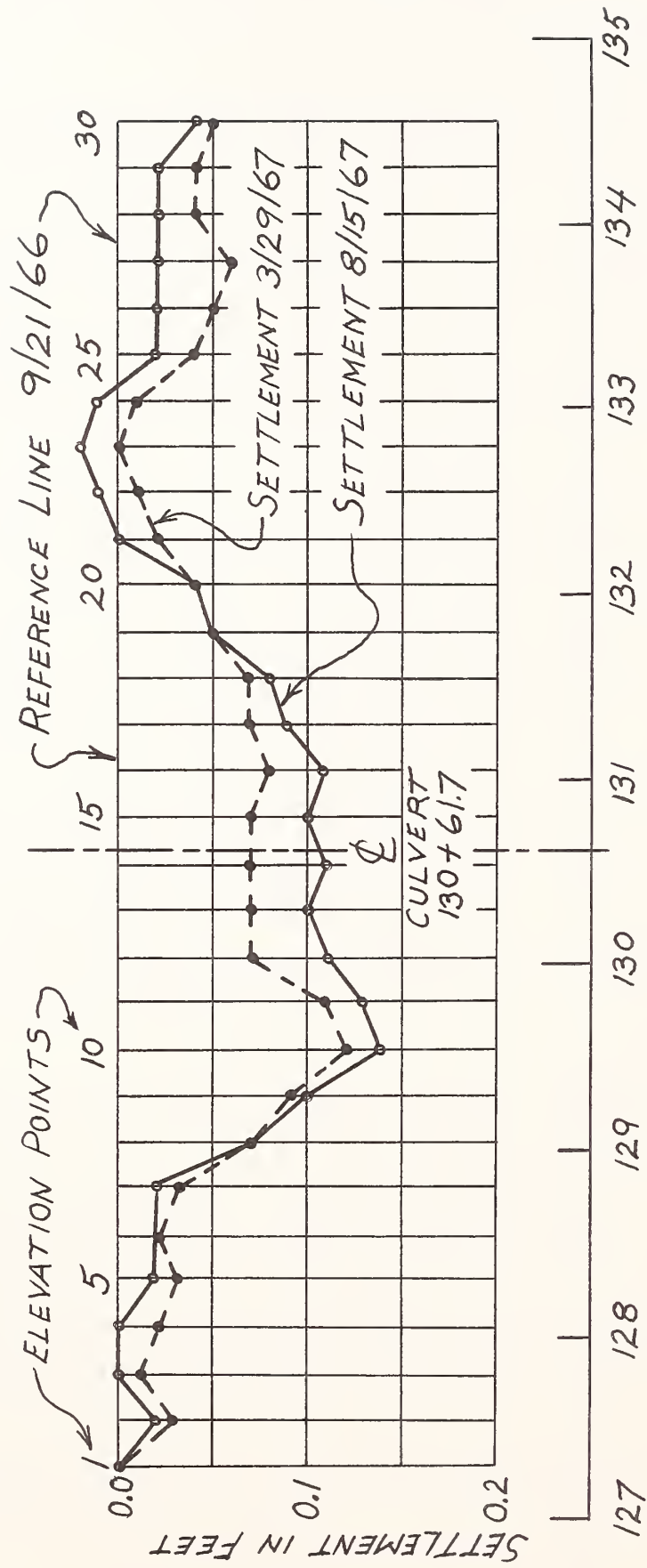


Figure 54. Settlement of the Interstate Roadway Pavement

side of the centerline guard rail.

Compression of the Straw Layer

During construction, no instrumentation was installed to permit routine measurement of the change in thickness of the three-foot thick layer of baled straw which formed the "imperfect trench". To obtain some factual knowledge of the amount of straw compression, a decision was made to cut a hole in the culvert roof and drill upward through the five feet of backfill and the straw to ascertain the compacted thickness of the straw. This was done on August 15, 1967, at ring No. 37 which was under the highest part of the embankment near the centerline of the highway.

A hole approximately six inches in diameter was cut in the culvert roof with a cutting torch and extended 5'-2" upward by hand to the base of the straw with a four-inch diameter post-hole auger. A hollow cylindrical rotary saw, fabricated from a discarded two-inch diameter propane tank, was then fastened to the end of the drill rods and turned by hand to cut a hole up through the straw. The entire operation was conducted by two men, from a wooden platform five feet below the culvert roof, and required about two days to complete. The rotary saw encountered a strand of baling wire about halfway through the straw and this proved to be a formidable obstacle for the makeshift saw but it was finally cut through after an hour or more of hard work.

The straw was dry and clean and showed no signs of deterioration

after almost two years of burial in the embankment. The granular backfill, between the straw and the top of the culvert, was hard and dry and showed no tendency to sluff or cave in. After the measurements were completed, the bottom part of the hole was plugged with stiff mortar for a length of about one foot.

At the site of this one exploration hole, the measured thickness of the straw layer was only 11.1". From direct measurements taken prior to covering up the straw, it was known that the layer was very nearly 36" thick initially. This means that the compression of the straw was 25".

If similar measurements had been taken at other representative stations, variations of several inches in the compressed thickness of the straw layer would undoubtedly have been found. It was felt, however, that the one measurement adequately established the order of magnitude of the straw compression and that the cost of trying to get a more refined estimate could not be justified.

The compressed thickness of the straw was used, along with observed settlement cell settlement, floor settlement, and vertical diameter change, to estimate the settlement of the "critical plane" at the top of the straw layer and apply Marston's Theory to calculate the vertical pressure on the straw.

Application of Marston's Theory to the Wolf Creek Culvert

M. G. Spangler's textbook "Soil Engineering" (24) is recommended as

a modern and readily available reference on Marston's Theory. Chapter 24 in that reference presents Marston's Theory for estimating loads on underground conduits, including trench or ditch conduits, projecting conduits, the "incomplete ditch condition" and the "imperfect ditch condition".

The rebuilt Wolf Creek culvert condition may be categorized as a "modified imperfect ditch condition" down to the bottom of the straw layer, with a reversion to a complete projection condition below the straw. This rather complicated case does not conform to any of the standard cases for which solutions are available to estimate the load on the culvert directly. However, we may imagine a fictitious "equivalent conduit" which includes the culvert, the straw, and the intervening earth, all three considered as a single unit. Marston's Theory may then be applied to this fictitious "equivalent conduit", which conforms to the so-called "incomplete ditch condition" pictured on page 405 of reference (24), to get an estimate of the vertical earth pressure acting on the straw.

Figure 55 illustrates the application of Marston's Theory to the Wolf Creek culvert. The height and width of the culvert were assumed as 19 feet and 18 feet, respectively (the culvert was constructed as an ellipse having an actual inside height of approximately 19'-4" and an inside width of approximately 17'-10"). The so-called "critical plane" is, in this case, the horizontal plane at the top of the straw layer just prior to covering up the straw with the remaining 75 foot depth of rockfill material.

The "settlement ratio", r_{sd} , may be calculated from the following relationship:

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m}$$

in which s_m = the vertical compression of the side columns of soil of height $p B_C$. This may be estimated with sufficient precision in this case as $\frac{27}{20}$ (ave. settlement cell settlement - s_g) $\cong \frac{27}{20} (3 \frac{1}{2} - 1) \cong 3 \frac{1}{2}$ ".

s_f = settlement of conduit into its foundation. This was measured as very nearly 1".

s_g = settlement of the natural ground surface adjacent to the conduit. This may be estimated with sufficient precision in this case as $s_g \cong s_f \cong 1$ ".

d_c = shortening of the vertical height of the conduit. For the equivalent conduit of Figure 55 this would be equal to the straw compression plus the vertical diameter change of the culvert, i.e., $d_c = 25" + 2" = 27"$. (The deformation of the earth between the straw and the culvert is negligible).

The calculated value of r_{sd} is then

$$r_{sd} = \frac{(3.5 + 1) - (1 + 27)}{3.5} = \frac{-23.5}{3.5} = -6.7$$

Also, it is seen from Figure 55 that the projection ratio, p , is equal to 1.5. The product of the settlement ratio and the projection ratio is then

$$r_{sd}p = -6.7(1.5) = -10.0$$

For all practical purposes this is equivalent to a complete ditch condition, as may be seen from the diagram on page 408 in reference (24).

Therefore, the diagram for ditch conduits, on page 401 in that reference, may be used to calculate the pressure on the straw. The curve for minimum pressure would come the closest to representing the correct condition for the cohesionless rock fill of the Wolf Creek culvert embankment. Then $H/B_c = 75/18 = 4.2$ which yields a value of $C = 2.1$ from the ditch conduit diagram. Estimating the embankment unit weight, w , as 125 pcf we may then estimate the pressure on the straw, according to Marston's Theory, as

$$\sigma = Cw B_c = 2.1 (125) (18) = 4700 \text{ psf or } 33 \text{ psi}$$

This result is almost double the average measured pressure of 17 psi on the base of the straw, as registered by the rubber diaphragm pressure cells, and it constitutes evidence that Marston's Theory is safe and conservative for estimating the vertical load on the straw in a "modified imperfect trench or ditch" situation which is similar in all essential respects to that used at the Wolf Creek site.

An error as large as a foot or more, in the measured thickness of the compressed straw, would not have had any effect on the answer obtained from Marston's Theory because a value of $r_{sd}p$ as small (numerically) as

minus 2 would still have indicated a situation equivalent to the complete ditch condition. Also, variations of several inches in either s_g or s_f would not change the settlement ratio enough to change the final answer. Even s_m , the most sensitive term in the settlement ratio formula, in this case, could be varied by a couple of inches without it having any effect on the final answer.

It is not surprising that the measured pressure on the straw was considerably less than that indicated by Marston's Theory, one reason being that the rock fill was placed above the straw in a fashion designed to maximize arching of the fill above the straw and thus minimize the pressure on the straw (see Figure 2). Before any rock was placed on the straw, it was built up and compacted on each side of the straw, on as steep a side-slope as possible, to a height of about 20 feet. An eight foot depth of loose uncompacted rock was then placed on the straw before any heavy earth-moving equipment was permitted to operate above the straw.

Another factor contributing to minimizing the pressure on the straw is the nature of the cohesionless rock fill. It is a reasonably dense graded mixture of angular rock fragments varying from dust-size all the way up to and including numerous large boulders of several cubic foot size. The large percentage of large angular fragments is particularly conducive to positive and permanent load relief through arching.

Arching of a fine-grained cohesive fill could sometimes be expected to

gradually "break down" through long term plastic flow but there is no need to worry about this in the case of the rock embankment above the Wolf Creek culvert. The load relief caused by arching above the straw may be considered permanent and independent of the future fate of the straw because the rock embankment will not experience a significant amount of long term plastic flow. As the straw rots away (if it ever does) the pressure on the straw may be expected to decrease progressively while leaving the load-relieving rock arch above permanently intact.

Pressure Increase from the Straw to the Culvert

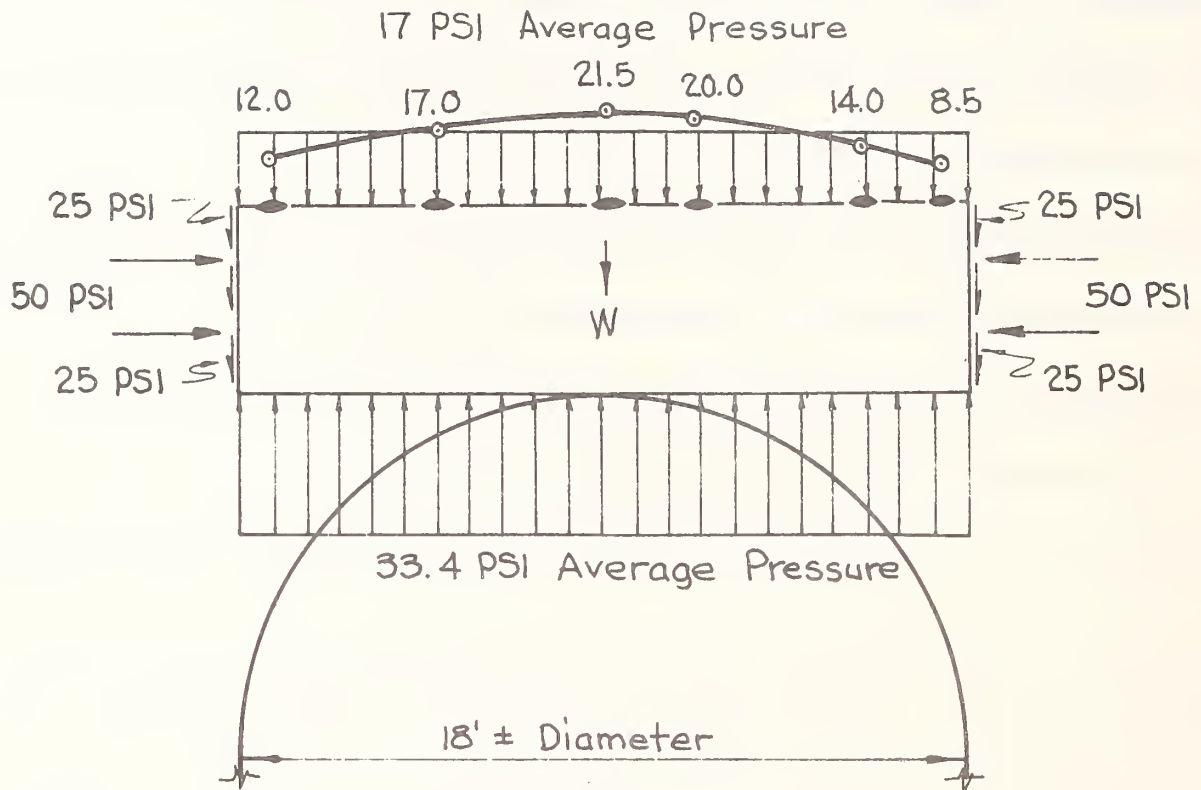
While Marston's Theory is adaptable for calculating a semi-rational estimate of the pressure on the straw, as demonstrated in the preceding section, it is not so readily adaptable for estimating the pressure increase between the straw and the culvert, where a complete projection condition exists with a major complicating factor in the form of a highly non-uniform pressure distribution on the plane labeled as the "critical plane" in Figure 55. The reader may refer back to Figure 50 to verify that the prisms of soil on each side of the straw carry vertical pressure in excess of 80 psi which is several times larger than the pressure on the straw. This fact invalidates any method based on treating the horizontal plane situated five feet above the culvert as though it were a plane of equal settlement.

Fortunately, it is a rather simple matter to make a direct and semi-

rational estimate of the pressure increase between the straw and the top of the culvert by merely drawing a free-body diagram of the prism of intervening soil and applying the principles of static equilibrium. This will be done to demonstrate the good correlation that exists between the pressures that were measured at the base of the straw and those acting directly upon the culvert. The pressures of January 4, 1966, will be used in the demonstration.

The rubber pressure cells were actually located six inches below the straw and 4 1/2 feet above the culvert; therefore, the free-body diagram used will be that of the 4 1/2 foot thick block of earth situated directly above the culvert. This is shown in Figure 56. An average vertical pressure of 17 psi, deduced from the rubber pressure cell results, is shown acting on top of the block. Also, a lateral pressure of 50 psi is shown acting on the sides of the block. Reference back to Figure 49 will show that 50 psi is a reasonable and probably not excessive estimate of the lateral pressure on the block.

To show that the adjacent earth is exerting downward frictional drag stresses on the sides of the block, reference is made to Figure 52 which shows that the settlement cells settled more than the top of the culvert. The settlement cells were located just a short distance off to each side of the central block. It may therefore be concluded that the adjacent soil settled downward relative to the central block and exerted downward frictional drag or shearing stresses on the sides. The 25 psi magnitude



Assume a Developed Internal Friction Coefficient of 0.5
 Shearing Stress = $50 \text{ PSI} \cdot 0.5 = 25 \text{ PSI}$

$$\text{Uniform Pressure due to Shearing Forces} \\ \frac{25 \text{ PSI} \cdot 4.5 \text{ Ft.} \cdot 12 \text{ In./Ft.} \cdot 2 \text{ Sides}}{18 \text{ Ft.} \cdot 12 \text{ In./Ft.}} = 12.5 \text{ PSI}$$

$$\text{Uniform Pressure due to Weight, W, of Soil below Rubber Cells} \\ \frac{125 \text{ PCF} \cdot 4.5 \text{ Ft.}}{144 \text{ Sq. In./Sq. Ft.}} = 3.9 \text{ PSI}$$

$$\text{Total Uniform Load} = 17 + 12.5 + 3.9 = 33.4 \text{ PSI at Top of Culvert}$$

Figure 56. Pressure on Top of Culvert Estimated from Rubber Pressure Cells.

shown for these stresses, which is just half of the 50 psi lateral pressure, is based upon an assumed developed internal friction coefficient of 0.5.

The calculations presented in Figure 56 show that the 17 psi pressure on the top of the block should be increased by about 12.5 psi to account for the effect of the frictional drag stresses, and an additional 3.9 psi to account for the weight of the block. This gives an estimated mean vertical pressure of about 33.4 psi at the top of the culvert.

Some engineers might choose to assume a developed friction coefficient as high as 0.84 which is equal to the tangent of 40 degrees; 40 degrees being a reasonable estimate of the true friction angle of the compact granular backfill material. The shear stresses on the sides of the block would then be taken as 42 psi and this would result in a pressure increase, due to the shearing stresses, of 21 psi instead of 12.5 psi. The estimated vertical pressure at the top of the culvert would then be 42 psi instead of 33.4 psi.

It was pointed out earlier (on page 93) that both the SR-4 strain gages and the soil stress meters indicated an average vertical pressure on the culvert of about 35 psi on January 4, 1966. This is in very good agreement with the 33 to 42 psi estimates based on the rubber pressure cell results. The 42 psi estimate is thought to be too high because it is based on a somewhat extreme estimate of the developed friction coefficient. On the other hand, downward friction drag forces exist below the level of the top

of the culvert; therefore, the total vertical load applied to the culvert is greater than what one might deduce by considering only the pressure at the level of the culvert crown.

CHAPTER IX

CLOSURE

Discussion and Design Implications

In summary it may be stated that every set of instrumentation at the Wolf Creek culvert site yielded plausible results and there was very good correlation among the various sets of instruments. For example, three independent sets of instrumentation; the SR-4 strain gages, the Carlson stress meters, and the rubber pressure cells in conjunction with measured settlements and deflections; all indicated that the maximum average vertical pressure on the culvert, which occurred in the winter of 1966 shortly after completion of the embankment, was in the neighborhood of 35 psi which is only approximately one-half of the nominal overburden pressure of 72 psi attributable to 83 feet of cover weighing 125 pcf.

One should not conclude from the figures above that it would have been appropriate or safe to design the culvert to carry only one-half the weight of the overburden because the lateral pressure, acting on the side of the culvert, was found to be substantially larger than the vertical pressure. One could reasonably infer, from the stress meter results of Figure 49, that the average lateral pressure on the culvert was at least 50 psi and perhaps more. The bending stress pattern revealed by the SR-4 strain gages, namely tension in the inside fibers of the sidewalls, is entirely consistent with a pattern of high lateral pressure and low verti-

cal pressure. Instrumented plates B and D actually demonstrated a significant increase in this type of bending (see Figures 39 and 40), which is consistent with an inward bulging of the sides and an upward bulging of the top, as the height of cover was increased from 5 feet to 83 feet. The fact that the horizontal diameter remains less than it was at the time of erection is also consistent with the observed pressure pattern. However, it should be printed out that the vertical diameter is also less than it was at the time of erection (see Figure 53) which indicates that a net outward bulging of the culvert walls has occurred somewhere between the sides and the top of the culvert.

In spite of the logical and consistent appearance of the results, the reader should recognize that the lateral pressure on the culvert was not measured with the same high degree of certainty as the vertical pressure. The Carlson stress meters on the sides of the culvert were the primary source of lateral pressure data and there were no SR-4 gages or rubber pressure cells to give a direct check on these lateral pressure results as there were for the vertical pressure.

The theory, expressed by some engineers, that the culvert might have a tendency to deflect upward into the compressible straw is lent some support by the high lateral pressure and the bending and deflection patterns referred to in the preceding paragraphs. Figure 53 shows that there has been a slight increase in vertical diameter since June of 1966. The tendency would have been more pronounced if the straw had been placed closer

to the top of the culvert but it is unlikely that a serious problem could have arisen because the compressed straw layer just isn't thick enough to permit a large amount of upward deflection. With the straw already compressed to a thickness of about 11 inches, any upward deflection of the culvert crown would have to be considerably less than 11 inches. Suppose the vertical diameter did increase by something like six inches; this would be less than a three percent deflection and would not be any cause for worry.

Ideally, the lateral pressure on a culvert should be equal to the vertical pressure or, more precisely, the pressure should be uniform over the entire exterior surface. It appears that a more nearly uniform pressure distribution would have been attained on the Wolf Creek culvert if the straw had been placed at a slightly higher elevation; perhaps about seven feet above the top of the culvert instead of five feet. Ostensibly this would have resulted in a somewhat larger vertical pressure on the culvert and a somewhat smaller vertical pressure on the columns of earth on each side of the culvert, which might have led to a reduction in the lateral pressure.

A less desirable pressure distribution than that attained would probably have occurred if the straw had been placed closer to the top of the culvert. The vertical pressure would surely have been smaller and it seems likely that the pressure difference between the top and the sides would have been accentuated.

As a practical matter, the five foot distance between the culvert crown

and the straw was not far from optimum and might well be copied at similar installations of about the same size.

The three foot unloaded thickness of the baled straw layer also proved to be very satisfactory. It compressed sufficiently to make the vertical pressure less than that based on Marston's complete ditch condition but it did not cause any measureable differential settlement of the pavement. Since the analysis based on Marston's Theory did indicate the equivalent of a complete ditch condition, between the straw and the top of the embankment, one should not rule out the possibility that some differential settlement of the top of the embankment did occur before the pavement was placed. All that the pavement profile levels proved was that there was no differential surface settlement, attributable to the straw, after the pavement was placed.

A culvert must resist the maximum load to which it is subjected, irrespective of the direction from which that load is applied. A culvert designed solely on the basis of an accurately predicted vertical load would be underdesigned if the horizontal load turned out to be larger than the vertical load, as it well might in the case of an installation utilizing a "modified imperfect trench" detail of the type reported herein. However, a safe design would still result if the "vertical design load" were calculated by a procedure sufficiently conservative to provide an answer at least as large as the actual maximum load, whether horizontal or vertical.

For the specific case of the rebuilt Wolf Creek culvert, this hypothetical procedure would have to provide an answer in the vicinity of 50 psi in order to agree with the measured lateral pressure.

Interestingly, a design pressure equal to 70 percent of the nominal 72 psi overburden pressure would amount to approximately 50 psi which appears to be just about "right" for the culvert as built, with the imperfect trench pressure relief detail included. One conventional design procedure for flexible highway culverts has been to design them to carry about 80 percent of the overburden weight; this appears to be a somewhat unconservative procedure unless a positive pressure relief detail is built in.

Throughout this report the rock embankment has been assumed to weigh 125 pounds per cubic foot and this is surely not too high. It appears to be reasonably well graded, with a maximum particle size of several cubic feet, and it would not have been unreasonable to have estimated the unit weight as high as 140 pcf. The reader should be aware that the true unit weight of the embankment may be somewhat greater than that assumed but it is not likely to be any less.

As mentioned earlier in the report, the pressure relief on the rebuilt Wolf Creek culvert may be considered permanent. The embankment and the foundation are both highly granular and cannot reasonably be expected to ever experience sufficient plastic flow to significantly reduce the beneficial arching. The same statement could not be made with the same high

degree of confidence if the embankment were made of clay. Designers should keep in mind that the pressure relieving effects of arching cannot always be considered permanent unless the arching material is predominately granular and non-plastic.

A review of Figures 42 through 51 shows that there has been some variation in pressure with time, following completion of the embankment. With few exceptions the measured pressures were largest shortly after completion of the embankment and then decreased for a period of six to ten months after which a rising trend occurred. In Figures 42 and 51 the rise was of small magnitude and leveled off again by August of 1967. Figures 44, 45 and 47 show some significant pressure increases but the pressures remained less than the early peak pressures.

Figure 43 is unique in exhibiting a gradually rising pressure, at the culvert crown, throughout the study period. It should be observed that the pressure increase of Figure 43 is entirely consistent with the slight upward deflection of the top of the culvert which occurred during the last year of the study and which may be seen in Figure 52.

Conclusions

Major findings of the study are summarized below in the form of a number of broad general conclusions:

1. Each set of instrumentation performed reasonably well and yielded plausible results which checked or correlated very well with other sets.

2. Erection of the structural plates , and placement and compaction of the backfill at the sides of the culvert , produced significant bending stresses in the culvert plates at the strain gage sites .

3. As the fill was brought up on each side of the culvert , the sides deflected inward and the top deflected upward . This action reversed as the remainder of the fill was placed ; however , the horizontal diameter never regained its original value and the vertical diameter decreased to a value slightly less than the original . During the last year of the study there was a very slight upward deflection of the top of the culvert .

4. In general , the load on the culvert reached its maximum value within a month or two following completion of the embankment , then decreased for six to ten months , and then increased somewhat but remained less than the earlier peak .

5. The demonstrated good agreement or correlation among the findings from all of the major sets of instrumentation proves beyond a reasonable doubt that the "imperfect trench" functioned in essentially the expected manner and caused the load on the culvert to be substantially less than the weight of the overlying fill . The highest load on the culvert is the lateral pressure exerted by the backfill on each side .

6. The roadway pavement , which was placed eight months after completion of the embankment , has not experienced any measurable differential settlement attributable to the "imperfect trench" .

Recommendations

This report leaves unanswered the highly relevant question: What would the pressure on the culvert have been if the pressure-relieving "imperfect trench" had not been used? A direct unequivocal answer to that question will never be possible but a similar and still more significant question, to which an answer can be found, is: What would be the behavior of a very similar culvert at a very similar site but with the "imperfect trench" omitted? It is recommended that a research project be undertaken to find out.

The performance of the SR-4 strain gages was especially good and it is recommended that more of them be used in any similar future investigation but that they be placed only on the inside walls. In retrospect it is clear that the strain gages placed on the outside of the Wolf Creek culvert could have been put to better use on the inside in spite of the fact that there is much theoretical merit in placing strain gages back-to-back. The same type of strain gages and lead wires, and the same manner of installation and waterproofing as reported herein, are highly recommended. One minor improvement would be to always file the zinc coating completely off and install the gages on bare steel. This procedure would eliminate any possible source of error that could arise from the zinc coating being poorly bonded to the steel.

It is recommended for future culvert studies that banks of strain gages be installed on the roof and floor, and at intermediate locations, as well as at mid-height, so that the bending behavior of culvert walls can be more fully investigated and so that the average pressure acting on a culvert from several different directions can be ascertained and reported in the manner illustrated in Figure 42.

Although the soil stress meters performed reasonably well, it is felt that the same amount of money and effort would return a larger dividend if spent on additional SR-4 installations. An earth pressure cell with the reliable operating characteristics of the Carlson Stress meter but without the protruding stem would be more easily adaptable to the job of accurately monitoring earth pressure at culvert sites.

The inexpensive rubber diaphragm pressure cells performed surprisingly well and are still giving highly stable and apparently reliable readings. This is in contrast to many of the strain gages and stress meters which deteriorated markedly in 1967 but not before they provided a wealth of valuable information. The deterioration has progressed to the point where little if any additional reliable information can be obtained from the strain gages and stress meters and it is recommended that the lead-wires be cut and removed so that they will not clutter up the inside of the culvert indefinitely. The walkways in the culvert, which were designed as temporary structures, will not remain safe for human use without continual maintenance.

It is therefore recommended that they also be removed.

The Bourdon gages connected to the still-functioning rubber diaphragm pressure cells should be left as is and read occasionally so that the curves of Figure 51 can be extended forward. It appears likely that the rubber pressure cells will continue to function reliably for several additional years. Pressure cells of this type have definite limitations and might not function well under sustained pressure in excess of 50 psi; however, they performed so well in this study that we recommend their use in future investigations wherever conditions appear suitable. A great many more of them would have been used at the Wolf Creek site if the project supervisor had been endowed with more faith in them at the outset. It is recommended that they be installed in the vicinity of other types of pressure cells so that each type can furnish an independent check on the other.

It is our firm conviction that earth pressure measurements are fraught with so many pitfalls and so many possible sources of error that several independent types of pressure transducers should be used together, whenever possible, so that each may serve to check the results of the others.

The settlement cells at the Wolf Creek site performed very well, after the researchers learned how to properly use them, and similar cells are recommended for future projects. The Wolf Creek installation should have been instrumented to permit routine measurement of the compression

of the straw. The settlement cells were considered for this job but rejected because of problems associated with installing a manometer board eight feet above the top of the culvert.

The researchers never did manage to think of it themselves, but they were told how it should have been done after the installation was completed. A steel plate with a vertical rod welded to its bottom face should have been placed on top of the straw, with the rod extending down into the culvert through a hole in the roof. The settlement of the plate could then have been readily deduced from the movement of the bottom of the rod. A pair of such plates, one on top of the straw and one at the bottom of the straw, could be used to precisely measure the change in thickness of the straw by merely noting the change in the vertical distance between the bottom ends of the two rods. It is recommended that several such pairs of plates be used on future installations.

A piece of unfinished business that is high on the project supervisor's priority list of future tasks, is to conduct a series of one-dimensional compression tests on bales of straw. If a pressure of about 20 psi compresses the straw bales to one-third of their original thickness it will constitute corroborative evidence that the rubber pressure cells accurately monitored the pressure at the base of the straw layer.

In the future, researchers should run many one-dimensional compression tests on the compressible material of their "imperfect trenches" and

establish the consolidation characteristics of that material accurately enough to enable them to deduce directly from its change in thickness what the pressure was that acted upon it to bring about that change in thickness. This simple method appears to have much merit and it should be tried to see if it works as well in the field as it does in the imagination.

Some types of underground structures, fallout shelters for example, could be completely surrounded by baled straw and the pressure everywhere on the structure could be estimated by the method outlined above. It should be kept in mind, however, that tangential drag stresses will act on the straw at the sides and distort it circumferentially so that radial distortion measurements will not suffice for deducing true radial pressure.

It would not be feasible to completely surround a highway culvert with straw but it might be feasible to backfill it with earth up to mid-height and then build a baled straw "igloo" over it. Presumably this would provide positive lateral pressure relief as well as vertical pressure relief. The straw could be stacked either directly against the culvert or against an interposed concentric layer of earth. It is recommended that research be conducted to determine the merits of this concept which seems to be a logical extension of the "imperfect trench".

APPENDICES

APPENDIX A
STRAIN GAGE COMPUTER PROGRAM
AND PARTIAL OUTPUT DATA

FORTRAN COMPUTER PROGRAM FOR DETERMINING THE STRESS, BENDING
MOMENT AND EARTH PRESSURE FROM THE STRAIN GAGES

```
C  C  STRESS CALCULATIONS, BENDING MOMENTS AND EARTH PRES-
      SURES FOR WOLF CREEK CULVERT
      DIMENSION ID(6), YINTAL(6),X(6), Y(6),ANAME(8)
      DIMENSION BARX(20), AREA(20), YSTR(20),XBAR(20)
1     FORMAT (3X, 12)
3     FORMAT (E14,8)
5     FORMAT (3X,12, 6X,E14.8, 6X,E14.8)
      READ 3,ELAST
      READ 20,(BARX(J), AREA(J), J=1,20)
20    FORMAT (2F10.6)
4     READ 2, ANAME
      KOUNT=0
2     FORMAT (8A4)
      PRINT 9, ANAME
9     FORMAT (1H1,8X,8A4)
      READ 1,N
      READ 40,CORP, CORM
40    FORMAT (F8.3,F11.3)
      PRINT 45,CORP
45    FORMAT (1HO,12X,33H APPARENT EARTH PRESSURE (PSI) = F15.3)
      PRINT 46,CORM
46    FORMAT (13X, 37H ERECTION STRESS MOMENT (IN-LB/IN) = F11.3)
      READ 5, (ID(I),YINTAL(I),X(I),I=1,N)
6     READ 7, IDATE1,IDATE2,IDATE3,TEMP,COVER,(Y(I),I=1,N)
7     FORMAT (3I2,3X,I3,I3,6E10.6)
C  C  KOUNT SEQUENCE TO GET THREE SETS OF DATA TO A PAGE
      KOUNT=KOUNT+1
      IF(KDATE1)4,99,88
88    IF(KOUNT-3)8,8,89
89    KOUNT=1
      PRINT 87,ANAME
87    FORMAT (1H1,8X,8A4)
8     DO 11 I=1,N
C  C  CALCULATE STRAIN AND DETERMINE STRESS
      Y(I)=Y(I)-YINTAL(I)
LL    Y(I)=Y(I)*ELAST
17    FORMAT (1HO,12X,8H DATE = I2,1H/,I2,1H/,I2,15H TEMPERATURE
1 = 13,9H 1 COVER = I3)
```

```

19  FORMAT (20X,3H G 12,6H X = F7.4,6H Y = F11.2)
    PRINT 17, IDATE1,IDATE2,IDATE3, TEMP,COVER
    PRINT 19, (ID(I),X(I), Y(I), I - 1,N)
C   C  DETERMINE THE SLOPE OF THE STRESS LINE BY THE METHOD OF
      LEAST SQUARES
      SUMSQ=0.0
      SUMX = 0.0
      SUMY=0.0
      SUMXY=0.0
      DO 140 I=1,N
      SUMSQ=SUMSQ + X(I)**2
      SUMX=SUMX+X(I)
      SUMY=SUMY+Y(I)
140  SUMXY=SUMXY+X(I)*Y(I)
      XN=N
      SM=((SUMY*SUMX)-(XN*SUMY))/((SUMX**2)-(XN*SUMSQ))
      SB=((SUMX*SUMXY)-(SUMY*SUMSQ))/((SUMX**2)-(XN*SUMSQ))
      PRINT 145,SM,SB
145  FORMAT (1HO,12X,11H SLOPE M = E14.8,15H INTERCEPT B =
1    E14.8)
      XX=1.1875
      YPOS = SM*XX+SB
      PRINT 150,YPOS
150  FORMAT (1HO,19X,24H STRESS AT SR4 GAGE 3 = 15X,F11.2)
      IF(ABSF(YPOS)-30000.)200,200,210
200  XX = -1.1875
      YNEG=SM**XX+SB
      PRINT 155,YNEG.
155  FORMAT (20X,25H STRESS AT SR4 GAGE 22 = 14X ,F11.2)
      IF(ABSF(YNEG)-30000.)205,205,210
205  XX=0.0
      YY=SM*XX+SB
      PRINT 160,YY
160  FORMAT (20X,39H STRESS AT CENTER SR4 GAGES 1 and 11 = F11.2)
C   C  DETERMINE BENDING MOMENT AND EARTH PRESSURE
      P=0.00433*YY-CORP
      YAVE=YNEG-YY
      YMOM=(YAVE)*0.197-CORM
      PRINT 206,P
206  FORMAT (1HO,12X,24H EARTH PRESSURE (PSI) = F8.3)
      PRINT 207,YMOM

```

```
207  FORMAT (13X,19H MOMENT IN-LB/IN = F13.3)
      GO TO 6
210  PRINT 211
211  FORMAT (1H0,12X,26H STRESS GREATER THAN YIELD)
C      C  DETERMINE BENDING MOMENT AND EARTH PRESSURE WHEN THE
      STRESS IS GREATER THAN THE YIELD STRESS
      YY=0.0
      XMOMT=-CORM
      FORCE=0.0
21  DO 30 J=1,20
      YSTR(J)=BARX(J)*SM+SB
      IF(YSTR(J)+30000.)33,29,31
31  IF(YSTR(J)-30000.)29,29,32
33  YSTR(J)=-30000.
      GO TO 29
33  YSTR(J)=30000.
29  XMOMT=XMOMT-BARX(J)*YSTR(J)*AREA(J)/3.
30  FORCE=FORCE+YSTR(J)*AREA(J)
      PRESS=FORCE/324.)-CORP
      PRINT 34, PRESS
34  FORMAT (1H0,12X,24H EARTH PRESSURE (PSI) = F8.3)
      PRINT 35,XMOMT
35  FORMAT (13X,19H MOMENT IN-LB/IN = F13.3)
      GO TO 6
99  CALL EXIT
      END
```


PARTIAL OUTPUT DATA FROM STRAIN GAGE COMPUTER PROGRAM

PLATE A

DATE=11/ 9/65 TEMPERATURE = 40 COVER = -18
G 22 X = -1.1875 Y = -21330.00
G 2 X = - .8125 Y = -14790.00
G 1 X = 0.0000 Y = - 1890.00
G 11 X = 0.0000 Y = - 1860.00
G 33 X = .8125 Y = 13920.00
G 3 X = 1.1875 Y = 21090.00
SLOPE M = .17799397E+05 INTERCEPT B = -.81000000+03
STRESS AT SR4 GAGE 3 = 20326.78
STRESS AT SR4 GAGE 22 = -21946.78
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 810.00
EARTH PRESSURE (PSI) = - 3.507 (Corrected to 0.0 for remaining
computations)
MOMENT IN-LB/IN = -4163.946 (Corrected to 0.0 for remaining
computations)

DATE= 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 22 X = -1.1875 Y = -32070.00
G 2 X = - .8125 Y = -22170.00
G 1 X = 0.0000 Y = - 3240.00
G 11 X = 0.0000 Y = - 3180.00
G 33 X = .8125 Y = 17820.00
G 3 X = 1.1875 Y = 27990.00
SLOPE M = .25071849E+05 INTERCEPT B = -.24750000E+04
STRESS AT SR4 GAGE 3 = 27297.82
STRESS AT SR4 GAGE 22 = -32247.82
STRESS GREATER THAN YIELD
EARTH PRESSURE (PSI) = - 7.103
MOMENT IN-LB/IN = -1647.712

DATE= 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 22 X = -1.1875 Y = -41160.00
G 2 X = - .8125 Y = -31260.00
G 1 X = 0.0000 Y = -12510.00
G 11 X = 0.0000 Y = -12180.00
G 33 X = .8125 Y = 9600.00
G 3 X = 1.1875 Y = 20490.00

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SLOPE M = .25698566E+05	INTERCEPT B = -.11170000E+05
STRESS AT SR4 GAGE 3 =	19347.04
STRESS AT SR4 GAGE 22 =	-41687.04
STRESS GREATER THAN YIELD	
EARTH PRESSURE (PSI) = -	39.959
MOMENT IN-LB/IN	= -1276.323

PLATE B

DATE= 11/ 9/65 TEMPERATURE = 40 COVER = -18
G 2 X = -.8125 Y = -7140.00
G 1 X = 0.0000 Y = - 540.00
G 3 X = 1.1875 Y = 7530.00
SLOPE M = .72954443E+04 INTERCEPT B = -.96193052E+03
STRESS AT SR4 GAGE 3 = 7701.40
STRESS AT SR4 GAGE 22 = -9625.27
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 961.93
EARTH PRESSURE (PSI) = - 4.165 (Corrected to 0.0 for remaining
computations)
MOMENT IN-LB/IN = -1706.678 (Corrected to 0.0 for remaining
computations)

DATE = 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 2 X = -.8125 Y = -12570.00
G 1 X = 0.0000 Y = - 1830.00
G 3 X = 1.1875 Y = 12330.00
SLOPE M = .12411429E+05 INTERCEPT B = -.22414285E+04
STRESS AT SR-4 GAGE 3 = 12497.14
STRESS AT SR-4 GAGE 22= -16979.99
STRESS AT CENTER SR-4 GAGES 1 AND 11 = - 2241.42
EARTH PRESSURE (PSI) = - 5.540
MOMENT IN-LB/IN = -1196.820

DATE = 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 2 X = -.8125 Y = -21300.00
G 1 X = 0.0000 Y = - 5640.00
G 3 X = 1.1975 Y = 12270.00
SLOPE M = .1666077E+05 INTERCEPT B = -.69725095E+04
STRESS AT SR4 GAGE 3 = 12811.33
STRESS AT SR4 GAGE 22 = -26756.35
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 6972.50
EARTH PRESSURE (PSI) = - 26.025
MOMENT IN-LB/IN = -2190.738

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PLATE C

DATE = 11/ 8/65 TEMPERATURE = 40 COVER = -18
G 2 X = -.8125 Y = 1860.00
G 1 X = 0.0000 Y = - 330.00
G 3 X = 1.1875 Y = -3870.00

SLOPE M = -.28735136E+04 INTERCEPT B = -.42081079E+03

STRESS AT SR4 GAGE 3 = -3833.10
STRESS AT SR4 GAGE 22 = 2991.48
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 420.81

EARTH PRESSURE (PSI) = - 1.822 (Corrected to 0.0 for remaining
computations)
MOMENT IN-LB/IN = 672.223 (Corrected to 0.0 for remaining
computations)

DATE = 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 2 X = -.8125 Y = -4350.00
G 1 X = 0.0000 Y = -2070.00
G 3 X = 1.1875 Y = 1470.00

SLOPE M = .29152124E+04 INTERCEPT B = -.20144014E+04

STRESS AT SR4 GAGE 3 = 1447.41
STRESS AT SR4 GAGE 22 = -5476.21
STRESS AT CENTER SR4 GAGES 1 AND 11 = -2014.40

EARTH PRESSURE (PSI) = - 6.900
MOMENT IN-LB/IN = -1354.200

DATE = 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 2 X = -.8125 Y = -9870.00
G 1 X = 0.0000 Y = -7500.00
G 3 X = 1.1875 Y = -3180.00

SLOPE M = .33664866E+04 INTERCEPT B = -.72708105E+04

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STRESS AT SR4 GAGE 3 =	- 3273.10
STRESS AT SR4 GAGE 22 =	-11268.51
STRESS AT CENTER SR4 GAGES 1 AND 11 =	- 7270.81
EARTH PRESSURE (PSI) =	- 29.660
MOMENT IN-LB/IN =	-1459.770

PLATE D

DATE = 11/ 8/65 TEMPERATURE = 40 COVER = -18
G 2 X = -.8125 Y = -6690.00
G 1 X = 0.0000 Y = 150.00
G 3 X = 1.1875 Y = 9570.00

SLOPE M = .81155216E+04 INTERCEPT B = -.44402164E+01

STRESS AT SR4 GAGE 3 = 9632.74
STRESS AT SR4 GAGE 22 = -9641.62
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 4.44

EARTH PRESSURE (PSI) = - .019 (Corrected to 0.0 for remaining
computations)
MOMENT IN-LB/IN = -1898.525 (Corrected to 0.0 for remaining
computations)

DATE = 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 2 X = -.8125 Y = -13020.00
G 1 X = 0.0000 Y = - 1470.00
G 3 X = 1.1875 Y = 15210.00

SLOPE M = .14109962E+05 INTERCEPT B = -.15237453E+04

STRESS AT SR4 GAGE 3 = 15231.83
STRESS AT SR4 GAGE 22 = -18279.32
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 1523.74

EARTH PRESSURE (PSI) = - 6.578
MOMENT IN-LB/IN = -1402.324

DATE = 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 2 X = -.8125 Y = -23040.00
G 1 X = 0.0000 Y = - 8160.00
G 3 X = 1.1875 Y = 17010.00

SLOPE M = .20110838E+05 INTERCEPT B = -.72438608E+04

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STRESS AT SR4 GAGE 3 =	16637.81
STRESS AT SR4 GAGE 22 =	-31125.53

STRESS GREATER THAN YIELD

EARTH PRESSURE (PSI) = -	31.357
MOMENT IN-LB/IN	= -2773.875

PLATE E

DATE = 11/ 9/65 TEMPERATURE = 40 COVER = -18
G 2 X = -.8125 Y = - 6870.00
G 1 X = 0.0000 Y = 330.00
G 3 X = 1.1875 Y = 8010.00

SLOPE M = .73686490E+04 INTERCEPT B = -.43108002E+03

STRESS AT SR4 GAGE 3 = 8319.18
STRESS AT SR4 GAGE 22 = -9181.35
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 431.08

EARTH PRESSURE (PSI) = - 1.866 (Corrected to 0.0 for remaining
computations)

MOMENT IN-LB/IN = -1723.803 (Corrected to 0.0 for remaining
computations)

DATE = 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 2 X = -.8125 Y = -13860.00
G 1 X = 0.0000 Y = - 1200.00
G 3 X = 1.1875 Y = 13710.00

SLOPE M = .13694826E+05 INTERCEPT B = -.21618532E+04

STRESS AT SR4 GAGE 3 = 14100.75
STRESS AT SR4 GAGE 22 = -18424.45
STRESS AT CENTER SR4 GAGES 1 AND 11 = - 2161.85

EARTH PRESSURE (PSI) = - 7.494

MOMENT IN-LB/IN = -1479.930

DATE = 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 2 X = -.8125 Y = -23760.00
G 1 X = 0.0000 Y = - 8970.00
G 3 X = 1.1875 Y = 5310.00

SLOPE M = .14350888E+05 INTERCEPT B = -.10933861E+05

STRESS AT SR4 GAGE 3 = 6107.81
STRESS AT SR4 GAGE 22 = -27975.54
STRESS AT CENTER SR4 GAGES 1 AND 11 = -10933.86

EARTH PRESSURE (PSI) = - 45.477
MOMENT IN-LB/IN = -1633.407

PLATE F

DATE = 11/ 9/65 TEMPERATURE = 40 COVER = -18
G 2 X = -.8125 Y = 3330.00
G 1 X = 0.0000 Y = -990.00
G 3 X = 1.1875 Y = -7830.00

SLOPE M = -.55932049E+04 INTERCEPT B = -.11308494E+04

STRESS AT SR4 GAGE 3 = -7772.78
STRESS AT SR4 GAGE 22 = 5511.08
STRESS AT CENTER SR4 GAGES 1 AND 11 = -1130.84

EARTH PRESSURE (PSI) = - 4.896 (Corrected to 0.0 for remaining
computations)
MOMENT IN-LB/IN = 1308.461 (Corrected to 0.0 for remaining
computations)

DATE = 12/ 2/65 TEMPERATURE = 41 COVER = 5
G 2 X = -.8125 Y = -3510.00
G 1 X = 0.0000 Y = -1140.00
G 3 X = 1.1875 Y = -360.00

SLOPE M = .15076448E+04 INTERCEPT B = -.18584554E+04

STRESS AT SR4 GAGE 3 = -68.12
STRESS AT SR4 GAGE 22 = -3648.78
STRESS AT CENTER SR4 GAGES 1 AND 11 = -1858.45

EARTH PRESSURE (PSI) = - 3.151
MOMENT IN-LB/IN = -1661.155

DATE = 1/ 4/66 TEMPERATURE = 2 COVER = 83
G 2 X = -.8125 Y = -12780.00
G 1 X = 0.0000 Y = -4980.00
G 3 X = 1.1875 Y = -3360.00

SLOPE M = .44645562E+04 INTERCEPT B = -.75980693E+04

STRESS AT SR4 GAGE 3 = -2296.40
STRESS AT SR4 GAGE 22 = -12899.72
STRESS AT CENTER SR4 GAGES 1 AND 11 = -7598.06

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EARTH PRESSURE (PSI) = 28.003
MOMENT IN-LB/IN = -2352.888

APPENDIX B
STRESS METER COMPUTER PROGRAM
AND PARTIAL OUTPUT DATA

FORTTRAN COMPUTER PROGRAM FOR DETERMINING THE TEMPERATURE AND
EARTH PRESSURE FROM THE STRESS METERS

```
C  C  TEMPERATURE AND EARTH PRESSURE CALCULATION FROM STRESS
      METER FOR WOLF CREEK CULVERT
      DIMENSION XTEMP(30),RO(30),XT(30),RTO(30),TEMP(30),
1  CORR(30),RR(30),RRO(30),XP(30),ID(30),PRESS(30)
      READ 5,N
      5  FORMAT (15)
      READ 10, (ID(I),RO(I),XT(I),RTO(I),RRO(I),XP(I),I=1,N)
10  FORMAT (12,5F7.2)
15  READ 20, IDATE1,IDATE2,IDATE3, COVER, (RR(I),XTEMP(I),I=1,N)
20  FORMAT (3I2,I3,5(F7.2,F6.2)/
1  (9X,F7.2,F6.2,F7.2,F6.2,F7.2,F6.2,F7.2,F6.2,F7.2,F6.2))
      IF(IDATE1)25,99,25
25  DO 30 I=1,N
C  C  DETERMINE TEMPERATURE
      TEMP(I)=(XTEMP(I)-RO(I)*XT(I)
C  C  DETERMINE TEMPERATURE CORRECTION
      CORR(I)=(XTEMP(I)-RTO(I))*XT(I)/1000.
      RR(I)=RR(I)-CORR(I)
C  C  DETERMINE THE EARTH PRESSURE
30  PRESS(I)=(RR(I)-RRO(I))*XP(I)
      PRINT 40,IDATE1,IDATE2,IDATE3, COVER
40  FORMAT (1H1,3(/1H0),16X,8HDATE=I2,1H/,I2,1H/,I2,10X,14H
1  COVER (FT) = 13)
      PRINT 45, (ID(I),TEMP(I),PRESS(I),I=1,N)
45  FORMAT (1H0,17X,13H CARLSON GAGE,2X,12H TEMPERATURE,2X,
1  15H EARTH PRESSURE/1H,22X,3HNO.,10X,6HDEG.F,10X,5H(PSI)
2  //(1H,I24,F16.0,F16.2))
      GO TO 15
99  CALL EXIT
      END
```


PARTIAL OUTPUT DATA FROM THE STRESS METER COMPUTER PROGRAM

DATE = 1/4/66

COVER (FT) = 83

CARLSON GAGE NO.	TEMPERATURE DEG.F	EARTH PRESSURE (PSI)
1	16.	22.12
2	11.	14.88
3	13.	25.07
4	12.	-15.03
5	10.	29.41
6	14.	23.08
7	14.	33.64
8	10.	35.98
10	9.	95.60
11	15.	18.05
12	9.	70.51
13	10.	72.15
14	10.	48.81
15	10.	43.09
16	17.	48.36
17	10.	19.51
18	12.	19.01
19	14.	46.45
20	11.	22.93
21	9.	19.76
22	14.	47.87
23	16.	23.23
24	15.	26.20
25	36.	88.27
26	33.	31.49
27	30.	10.99
28	22.	47.07
29	32.	5.86
30	35.	80.70

DATE = 11/ 1/66

COVER (FT) = 83

CARLSON GAGE NO.	TEMPERATURE DEG. F	EARTH PRESSURE (PSI)
1	38.	23.42
2	36.	16.37
3	37.	.40
4	36.	-77.61
5	36.	7.45
6	35.	23.78
7	37.	40.45
8	36.	23.56
10	36.	58.78
11	38.	25.69
12	36.	38.62
13	36.	84.88
14	37.	25.10
15	37.	12.00
16	38.	15.48
17	37.	5.85
18	40.	-11.35
19	40.	7.96
20	38.	4.56
21	37.	6.83
22	40.	23.18
23	41.	33.92
24	40.	- .74
25	49.	103.60
26	51.	.56
27	52.	2.01
28	48.	10.12
29	52.	- 1.65
30	50.	26.83

DATE = 3/29/67

COVER (FT) = 83

CARLSON GAGE NO.	TEMPERATURE DEG.F	EARTH PRESSURE (PSI)
1	38.	31.35
2	37.	18.78
3	38.	1.07
4	36.	-74.65
5	36.	4.94
6	36.	26.19
7	36.	45.84
8	36.	28.44
10	36.	62.49
11	37.	31.14
12	36.	52.73
13	36.	98.35
14	36.	33.10
15	36.	15.58
16	37.	15.44
17	37.	10.11
18	37.	5.70
19	38.	13.47
20	37.	9.22
21	36.	9.71
22	38.	30.00
23	38.	29.14
24	38.	2.10
25	39.	104.18
26	35.	.85
27	37.	2.92
28	38.	16.44
29	36.	-3.72
30	37.	21.65

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